HUMBER SEWERSHED

COMBINED SEWER OVERFLOW STUDY

STANDARDS DEVELOPMENT BRANCH OMOE
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TECHNICAL REPORT # 7

A REPORT

OF THE

TORONTO AREA WATERSHED
MANAGEMENT STRATEGY
STEERING COMMITTEE

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HUMBER SEWERSHED
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TECHNICAL REPORT #7

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TORONTO AREA WATERSHED
MANAGEMENT STRATEGY
STEERING COMMITTEE

Prepared by:
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Water Resources Branch
Ontario Ministry of the Environment

June, 1986

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A study of this nature and scale is necessarily a team effort. The author's many thanks are due to all the participants and supporters involved.

SUMMARY AND CONCLUSIONS

1.0 Study Objective

The objective of the study is to provide the Toronto Area Watershed Management Strategy Study (TAWMS) with information concerning combined sewer overflows (CSO) to the Humber river and possible control of the CSO pollution. TAWMS plans to use the information for the development of a comprehensive water quality management strategy for the Humber river.

2.0 Study Area Characteristics

2.1 The study area comprises a combined sewer area and a sanitary sewer area in Metro Toronto in the Humber river watershed. The two areas share the same Metro sewer and the Humber Water Pollution Control Plant (WPCP) for conveyance and treatment of their sewage. By including the sanitary sewer area in the study, the total hydraulic loading on the Metro facilities can be determined in the case of CSO control measures affecting these facilities.

The following are basic data of the two sewer areas under existing (1983) conditions:

	Combined Sewer Area	Sanitary Sewer Area
Area	1,246 ha	17,418 ha
Population	81,000	449,000
Dry-weather flow	36,000 m^3/d	320,000 m^3/d

The combined sewer area is 7% of the study area.

2.2 In wet weather, combined sewage up to 2.43 m^3/s (5.8 times of dry-weather flow of the combined sewer area) is allowed to flow to the WPCP. Flow in excess of this limit is partly diverted to an existing detention tank (Hyde Avenue tank) and the remainder

overflows to the Black Creek (a tributary to the Humber river) via 3 regulators set in the sewers. The tank has a capacity of 7,800 m^3. When full, the tank also overflows to the Black Creek. The detained flow is returned to the WPCP after a storm.

2.3 Monitoring results in April-October, 1983 indicate that the regulators overflow to the Black Creek if the storm has more than 4 mm precipitation. The smallest storm in which the existing tank is filled has 7.4 mm precipitation.

The observed, flow-weighted average pollutant concentrations of the combined sewage are as follows:

	mg/l		mg/ l
Suspended solids	196	BOD_5	55
Total phosphorus	1.96	Soluble phosphorus	0.44
Lead	0.182	Zinc	0.300
Copper	0.119	Cadmium	0.006

Fecal coliforms (counts/100 ml) 1.65×10^6
The results do not show any abnormal concentrations.

2.4 In addition to the above-mentioned regulators, an overflow regulator (near Berry Road) exists in the Metro trunk sewer just upstream of the WPCP. Monitoring results of this regulator in June-October, 1984 show that it overflows sparingly in a thunderstorm only and the overflow duration is about 1 hour. This observation supports the model simulation results in paragraph 4.2 that this regulator overflows only in intense storms.

2.5 According to the original estimate of the Metro Toronto authority, the WPCP has peak capacities of 11.8 m^3/s (primary treatment and outfall) and 9.6 m^3/s (secondary treatment). The authority has now revised the capacities of both primary and secondary treatment to 8.9 m^3/s. All the capacities are hypothetical values. It appears that an in-WPCP evaluation is

necessary if the true capacities are to be determined. Subject to the further investigation, this study assumes the capacities as originally estimated. The findings in the report are not affected by the revised capacities except that CSO Control Scheme 2 (paragraph 5.0) will not be feasible if the peak primary capacity should, in fact, be less than 11.8 m^3/s.

3.0 Analysis

Model simulation is used to estimate CSO statistics for existing and postulated conditions of catchments and sewers. All analyses use the precipitation data of the April-October 1979 season which is found to be representative of the average conditions in the precipitation history of the study area.

4.0 The Existing CSO Situation

4.1 With catchments and sewers in existing (1983) conditions, the regulators overflow in 26 storms, excluding the intense storm mentioned in paragraph 4.2. There are 64 rain events in the season. The estimated seasonal total volumes and pollutant loads, also excluding the intense storm, are as follows:

	Overflow*	Detained by Tank
Volume (m^3)	334,000	102,000
Suspended solids (kg)	63,000	16,000
BOD_5 (kg)	16,000	5,000
Total phosphorus (kg)	690	210
Soluble phosphorus (kg)	230	72
Lead (kg)	65	20
Zinc (kg)	112	35
Copper (kg)	41	13
Cadmium (kg)	2.6	0.8

(* includes overflow from existing tank)

The overflow volume is 20% and the detained volume 8% of the combined sewage yielded by the combined sewer area in the 26 storms.

Seasonal fecal coliform load is not given because it is a meaningless statistic. In a single storm, CSO fecal coliform load ranges from 80,000 billion to 670,000 billion organisms.

If CSO in the 26 storms is intercepted and treated at the Humber WPCP, the increase in WPCP seasonal treatment load will be 1/2%.

The impacts of CSO pollutant loads on the receiving waters will be studied by a separate TAWMS project.

4.2 The intense storm mentioned in paragraph 4.1 has a recurrence interval of 3.3 years, a total precipitation of 36.4 mm, and a maximum 1-hour precipitation of 28.1 mm. It produces a total overflow volume of 265,000 m^3, of which 160,000 m^3 overflow from the combined sewer area to the Black Creek and 105,000 m^3 overflow from the Berry Road regulator to the Humber river. The Berry Road overflow is attributed to wet-weather inflow/infiltration from the sanitary sewer area, not sewage from the combined sewer area. This is the only storm that causes the Berry Road regulator to overflow. The largest single-event overflow volume in all the other storms in the season is 41,000 m^3 only. It is concluded that the storm that causes the Berry Road regulator to overflow is intense and infrequent enough to be excluded from consideration in the development of CSO control schemes.

4.3 Under existing conditions, the WPCP has sufficient peak capacity to treat the wet-weather flow it receives. The combined sewer area requires a larger peak treatment capacity (0.0000298 m^3/person/s) than the sanitary sewer area (0.0000164 m^3/person/s). Their ratio is 1.8 to 1.0. Averaged over the season, however, the combined sewer area requires a smaller average unit treatment capacity than the sanitary sewer area. The respective figures are 0.526 m^3/person/d (including combined sewage) and 0.735 m^3/person/d. Their ratio is 1.0 to 1.4.

5.0 CSO Control Schemes

5.1 Five CSO control schemes are considered feasible. Each scheme is self-contained and is an alternative to the other four. Each scheme is analyzed for its effectiveness in CSO reduction against various capacities up to complete CSO elimination in the season (except in the intense storm). The maximum recurrence interval of storms in which the schemes may achieve complete CSO elimination is 1.8 years. The schemes are listed below in descending order of their relative merits.

		Order of Costs for Complete CSO Elimination $ million
1.	Detention of overflow at regulators.	2.8
2.	Increase in combined sewage flow to WPCP by resetting regulators.	3.5
3.	Schemes using runoff control:	
	(i) Detention tanks in local combined sewers.	6.1
	(ii) Disconnection of existing roof leaders.	3.1
	(iii) Separation of combined sewers.	14.0

5.2 General notes on the schemes:

- The components of each scheme are defined in paragraph 5.3.

- Costs shown exclude costs of land, engineering services and ancillary works such as access road, instrumentation and landscaping of storage tank sites. Detailed engineering feasibility and costing will be studied by a separate TAWMS project.

- Flow detained in storage will be returned to the WPCP after a storm.

- Land for new storage tanks at the regulators is now zoned as green space. Change in use in some land is expected but land required for new storage is expected to be available.

- Stormwater runoff in each of Schemes 3(i), (ii) and (iii) is reduced by an equivalent of 20% of the combined sewer area. Schemes assuming more reduction (28%) of the combined sewer area are studied but the stormwater runoff reduced for each hectare of area reduced is not better than the results of Schemes 3(i), (ii) and (iii).

5.3 Comments on individual scheme:

- All capacity requirements given below refer to complete CSO elimination as defined.

Scheme 1: Detention of overflow at regulators.

- Scheme consisting of 2 new tanks. One tank (16,000 m^3) to intercept overflow from existing tank; one tank (35,000 m^3) to intercept all regulators overflowing to Black Creek.

- Cheapest, most practical, most reliable scheme.

Scheme 2: Resetting regulators.

- Scheme consisting of duplication of existing Metro Black Creek sanitary trunk sewer by a new sewer of 1.2 m diameter and 2.1 km length and 2 new tanks (16,000 m^3 and 15,000 m^3) at regulators.

- Regulators to be reset to increase wet-weather flow to WPCP from 2.43 to 4.43 m^3/s.

- Peak primary treatment capacity of WPCP assumed to be 11.8 m^3/s. Scheme not feasible if peak primary treatment capacity to be less than 11.8 m^3/s.

- The increased wet-weather flow to WPCP to receive primary treatment only.

- WPCP capacity stressed to the limit in wet weather.

- Possible disruption of golf course by construction of new sewer.

Scheme 3(i): Detention tanks in local combined sewers.

- Scheme consisting of 50 tanks of 300 m^3 each in local sewers and 29,000 m^3 of new storage at regulators.

- Systematic program for cleaning and maintenance of flow valves in local tanks required.

- No apparent serious disadvantage except high cost.

Scheme 3(ii): Disconnecting roof leaders.

- Scheme consisting of disconnection of roof leaders of 5,800 houses and provision of 29,000 m^3 of new storage at regulators.

- Scheme considered marginally feasible. Effectiveness of roof leader disconnection in flow reduction not well proven.

- No provision in existing sewer by-law for disposition of roof leaders, except in special cases.

- Public acceptance of large scale disconnection uncertain.

- Possible seepage of surface water into basements if disconnection done improperly.

Scheme 3(iii): Separation of combined sewers.

- Scheme consisting of new sewers in 248 ha of catchment for separation of road drainage from combined sewers and 29,000 m^3 of new storage at regulators.

- Extensive disruption of neighbourhood by sewer construction.

- Many years required for completing a sewer separation programs. Meantime, partially completed sewers not able to function fully.

- Runoff pollutants of the separated area still discharged to receiving waters. Seasonal SS load so discharged estimated at 67,000 kg, slightly more than the 63,000 kg of CSO SS load before sewer separation.

- Stormwater discharges from the new storm sewers not contaminated by sanitary sewage.

6.0 Basement Flooding Mitigation

A cursory analysis is carried out to explore the possibility of integrating some measures for basement flooding mitigation into CSO control schemes, although the analysis is outside the study scope. The results indicate that design conditions for flooding mitigation and CSO control are fundamentally different and the measures for the two objectives have to be provided independently of each other. The provisions for the two objectives do not augment each other.

RECOMMENDATIONS

1. TAWMS should ascertain whether or not CSO control should be included in the watershed management plan.

2. If CSO control is required solely for bacterial control and not for other CSO parameters or pollutants, then TAWMS should consider whether high-rate disinfection can be a viable alternative to the CSO control schemes presented in this report.

3. The five alternative CSO control schemes should be considered in the following descending order of preference, subject to detailed feasibility study and costing:

   - Detention of overflow at regulators
   - Resetting regulators
   - Detention tanks in local sewers
   - Disconnecting roof leaders
   - Separation of combined sewers.

4. CSO control in less frequent storms (recurrence interval longer than 2 years) is not recommended.

5. Scheme 3(ii) using roof leader disconnections should be considered for adoption only after its effectiveness and reliability are proven by a large-scale pilot project.

6. The implementation of a CSO control scheme should be phased, preferably as follows:

Scheme	Phase 1	Phase 2
- Detention of flow at regulators	Tank to intercept regulators	Tank to intercept Hyde Avenue tank
- Resetting regulators	Tank to intercept regulators	1. Black Creek sewer duplication 2. Tank to intercept Hyde Avenue tank
- Detention tanks in local sewers	Tanks at 1st priority locations	Tanks at 2nd priority locations
- Disconnecting roof leaders	Depending on institutional considerations	
- Separation of combined sewers	Downstream sections of sewers	Upstream sections of sewers

7. The implementation of a CSO control scheme should be accompanied by a performance monitoring program. The program should be planned scientifically and supervised properly.

8. The incompatibility of using a CSO control scheme to augment basement flooding mitigation or vice versa should be recognized.

1.0 INTRODUCTION

1.1 Background

The Combined Sewer Overflow Study presented in this report is one of the projects of the Toronto Area Watershed Management Strategy Study (TAWMS). The goal of TAWMS is to develop a comprehensive water quality management strategy for the Humber and the Don river watersheds (Figure 1.1).

Initial emphasis of TAWMS is on the Humber river watershed. This report will mention work done on this watershed only.

The Humber river watershed has a drainage area of about 897 square kilometres (Acres, 1984). Sub-watersheds in the upper reaches of the river are primarily rural or agricultural areas (Figure 1.1). Sub-watersheds in the lower reaches are highly urbanized and include Metropolitan Toronto.

Water qualities of the river were studied in a TAWMS project (Ministry of the Environment, 1983). Its conclusions indicated that:

- bacterial densities in the urbanized reaches were high and body-contact recreation often had to be restricted;

- there was continual enrichment of stream waters;

- some heavy metals, including cadmium, copper, lead and zinc, were often found at concentrations in excess of provincial water quality objectives; and

- the Black Creek, which is a tributary to the Humber river (Figure 1.1), showed the most degraded water quality.

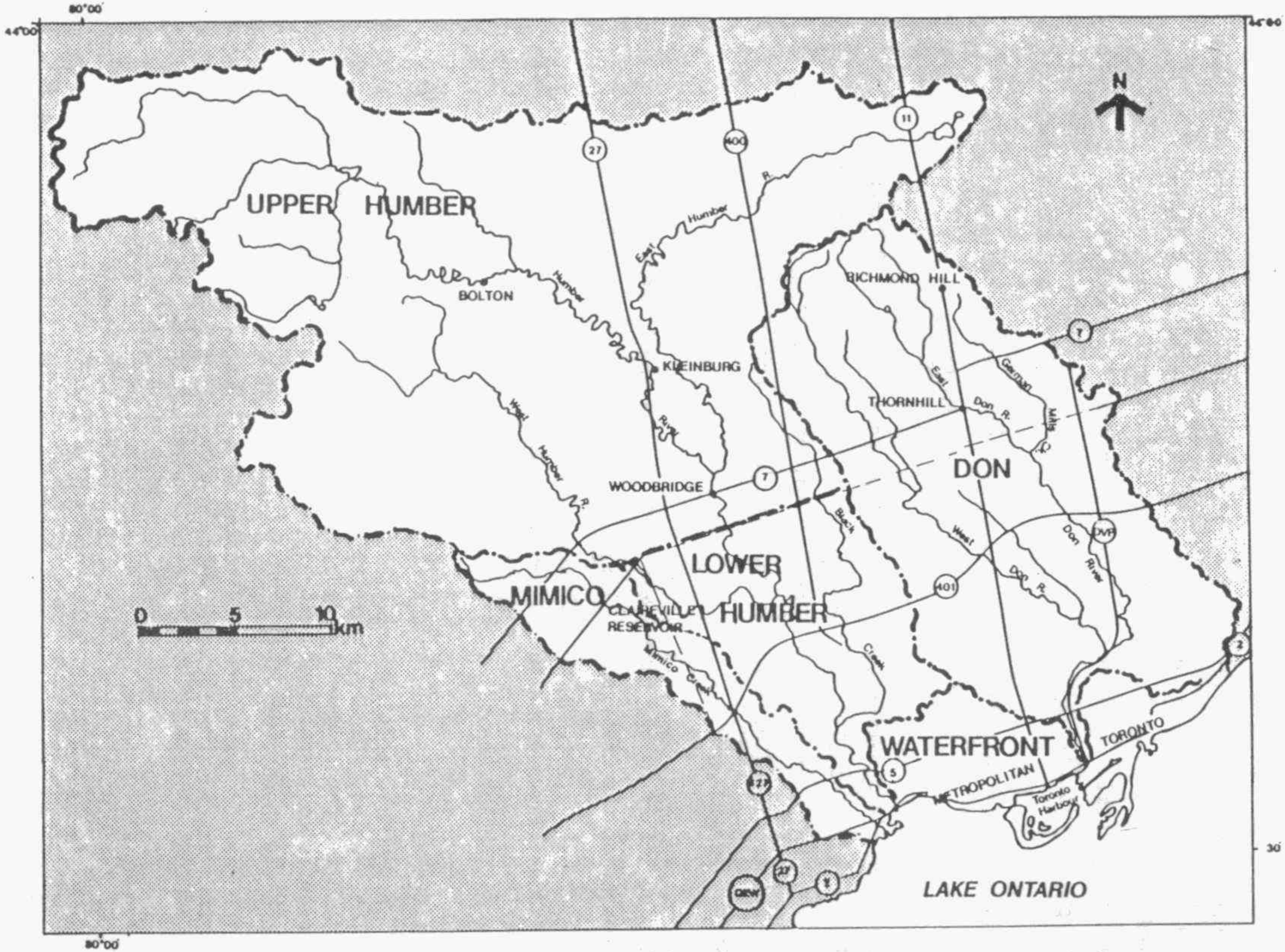


FIGURE 1.1: TORONTO AREA WATERSHED MANAGEMENT STRATEGY STUDY

In the light of the findings of the above preliminary study, TAWMS takes the following 3-part approach in pursuit of its goal:

- To investigate pollution sources and explore and evaluate means of controlling them;

- To examine impacts of pollution sources on the water qualities of the Humber river and predict water quality improvements due to pollution controls; and

- To develop a strategy for management of the water qualities of the river. The strategy will be developed using results of the investigations and having regards for economic, social, environmental and institutional effects of the strategy.

The Combined Sewer Overflow Study (CSO study) belongs to the first part of this approach. It started in 1983 after some preliminary field monitoring work was carried out in late 1982.

1.2 Organization of Report

The text of this report is written for a general readership. Technical matters are presented as simple as possible in the text. More complex technical discussions and details are placed in appendices.

The text, however, includes some introductory discussions that provide an underlying understanding of the nature, analysis and control of combined sewer overflow. The text also includes some literature review to illustrate how combined sewer overflow studies were carried out elsewhere in recent years. The purpose of the literature review is to share experiences of others and to provide a measure of the technical adequacy of the present CSO study. Readers may pass over those topics, if they so wish, without loss of continuity of the text.

1.3 Introductory Discussion of Combined Sewer Overflow

Combined sewers were laid in many urban areas developed before the 1960's to collect and transport both sanitary wastewater and stormwater surface runoff. Newer urban areas are served by two separate sewer systems, one for sanitary wastewater and one for stormwater runoff.

In a combined sewer, the dry-weather flow is basically sanitary wastewater but in wet weather, the flow is a mixture of both sanitary wastewater and stormwater runoff. This mixture is combined sewage. Its occurrence is intermittent and coincides with the occurrence of stormwater runoff. In the Toronto area, wet weather in the summer occurs at an average interval of about 3 days.

The flow rate of combined sewage varies from moment to moment in a storm and so does the proportion of sanitary wastewater and runoff in the flow. Figure 1.2 is an observed example demonstrating the variations in the flow rate and flow proportion. It can be seen that the combined sewage flow rate can be many times the dry-weather flow rate.

In a combined sewer network, provision often exists for some sewage to escape or overflow from the sewers via regulators under certain high flow conditions.

Regulated overflow is a means of ensuring that the water pollution control plant that intercepts the sewers will not be overloaded in wet weather. It is also a means of releasing excessive flow in an "orderly" manner to avoid sewage backing up in sewers with the result that sewage overflows indiscriminately from manholes and into basements of buildings. It is common practice to discharge the overflow to a watercourse.

An overflow regulator is a specially designed opening usually made at the side of a sewer. Its shape and level of setting determine the sewer flow rate (the threshold capacity) at which overflow will

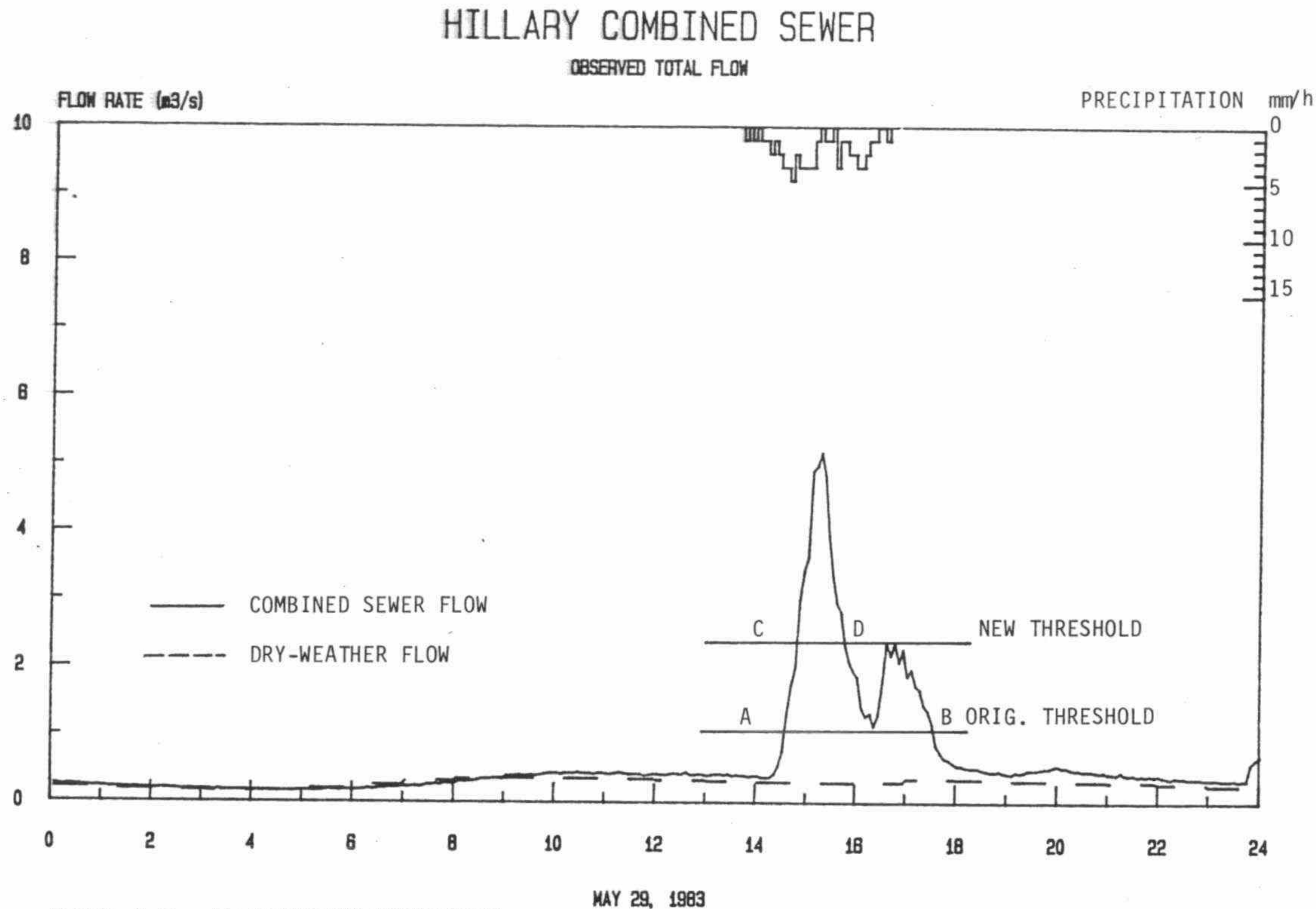


FIGURE 1.2 : ILLUSTRATIVE HYDROGRAPH

occur. Again using Figure 1.2 for illustration, if the threshold capacity is 1.0 m^3/s, overflow will occur between 14:30 hr. and 17:30 hr. and the volume of overflow is the area bounded by the curve and the line AB drawn at the threshold capacity. If the threshold capacity is increased to, say, 2.5 m^3/s, the overflow duration will be shortened to between 14:40 hr and 15:40 hr. The overflow volume will be the area bounded by the curve and the line CD. It is smaller than the previous volume.

A regulator is usually designed with an overflow capacity such that the sum of the overflow capacity and the threshold capacity is at least equal to the capacity of the sewer leading to the regulator. This design approach is to ensure that the regulator will not become a flow constriction to cause surcharge of the sewer upstream of the regulator.

The study of combined sewer overflow is closely related to the study of stormwater runoff and the sewer network. One major difference between a stormwater study and a combined sewer overflow study is that combined sewer overflow contains sanitary wastewater. Therefore, for some pollutants such as pathogenic bacteria, combined sewer overflow is a more potent polluter than runoff alone. Another major difference is that, while a storm produces runoff, the storm does not necessarily produce sufficient combined sewage to cause overflow.

2.0 OUTLINE OF STUDY

2.1 Study Objective and Scope

The CSO study was initiated with the objective of providing TAWMS with CSO information of the Humber sewershed for the development of TAWMS's water quality management strategy for the watershed.

The CSO study area is shown in Figure 2.1. The required scope of the study comprises the following:

1. To collect data required for the CSO study; snow data not required;

2. To characterize the catchments and sewers of the CSO system (the study system);

3. To estimate CSO pollution from the existing study system; and

4. To develop possible schemes for control of the CSO pollution and to evaluate the effectiveness of the schemes in CSO reduction.

Moreover, the study requires that computer model simulation will be employed in the CSO analysis. The analysis work will be carried out with sufficient detail to produce CSO statistics to meet the study objective, but hydraulic analysis of local trunk sewers will not be required. Pollutants to be studied will be suspended solids, total and soluble phosphorus, lead, zinc, copper, cadmium, fecal coliforms and BOD_5. The CSO analysis period will be a selected season of a selected year. The season will be selected in which CSO is typically most predominant in the year. The year will be selected in which precipitation will be representative of historical average conditions. All CSO control schemes will be subject to two constraints, namely, there will be no increase in sewage level (hence no aggravation of basement flooding) in the sewer network upstream of overflow regulators; and the peak capacities of the

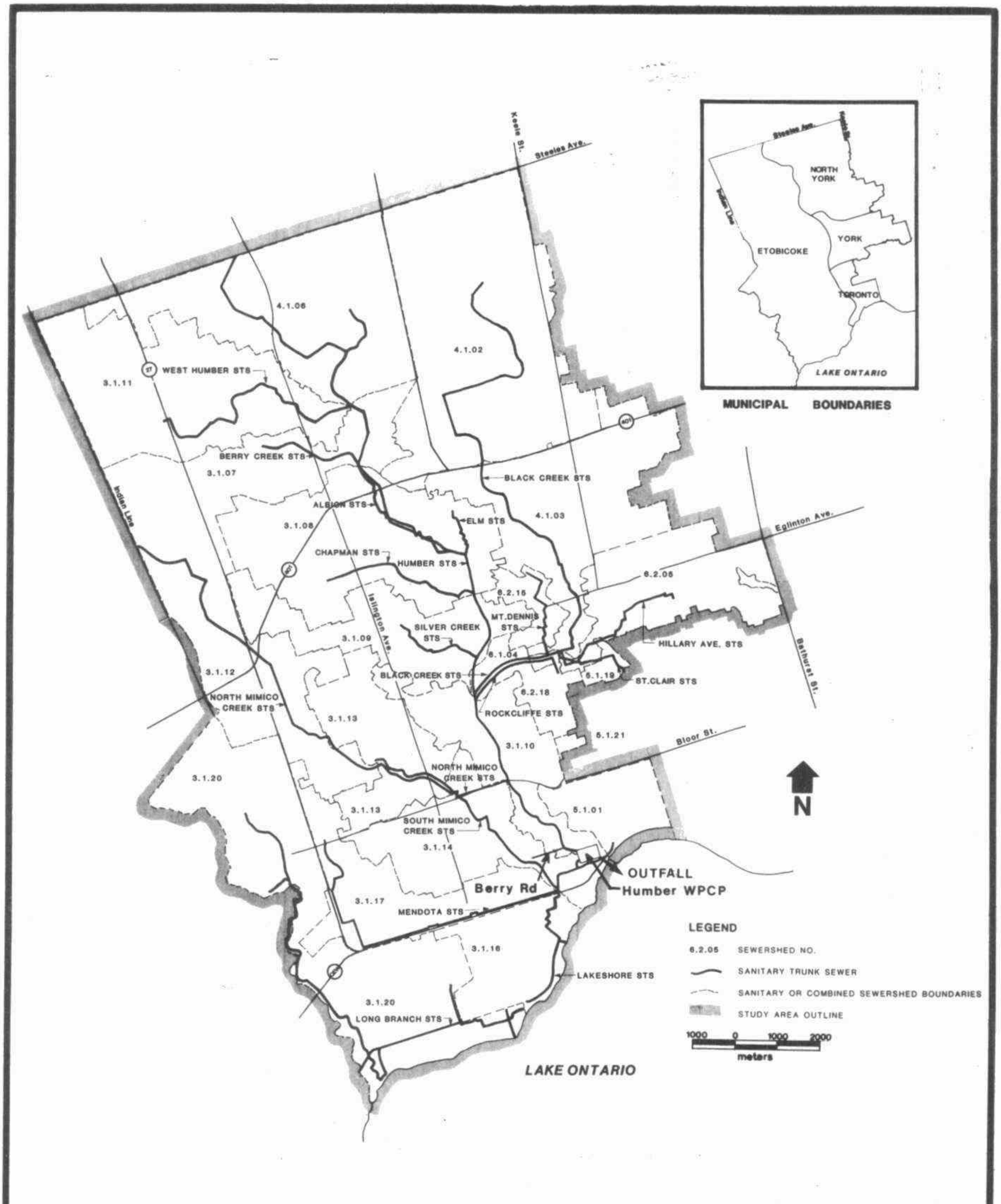


FIGURE 2.1 : HUMBER SEWERSHED : SANITARY AND COMBINED SEWER KEY MAP

existing Humber Water Pollution Control Plant (WPCP) will not be exceeded. CSO control schemes will be developed to the conceptual layout stage so that engineering feasibility and costs of the control schemes may be studied by a separate TAWMS project.

2.2 Overview of the Study

The action plan developed for the CSO study is outlined in Figure 2.2 and briefly described below.

Activity	Purpose
Collect catchment and sewer data	To understand the study system. To configure the system for hydrologic, hydraulic and pollutant loading computations.
Collect flow data	To observe response of the study system to wet and dry weather. To derive parameter values for estimating flow and pollutant loadings. To provide observed data to validate simulation results.
Collect precipitation data	To provide a link between observed precipitation and observed flow.
Model simulation	To obtain methodically statistics and trends of CSO loading changes in response to control.
Provide results for water quality study	To provide input and link for evaluating CSO impacts on receiving waters.
Draw up conceptual control schemes	To indicate conceptual layout of CSO control schemes for examining feasibility and costs and for future reference for design and operation of schemes.

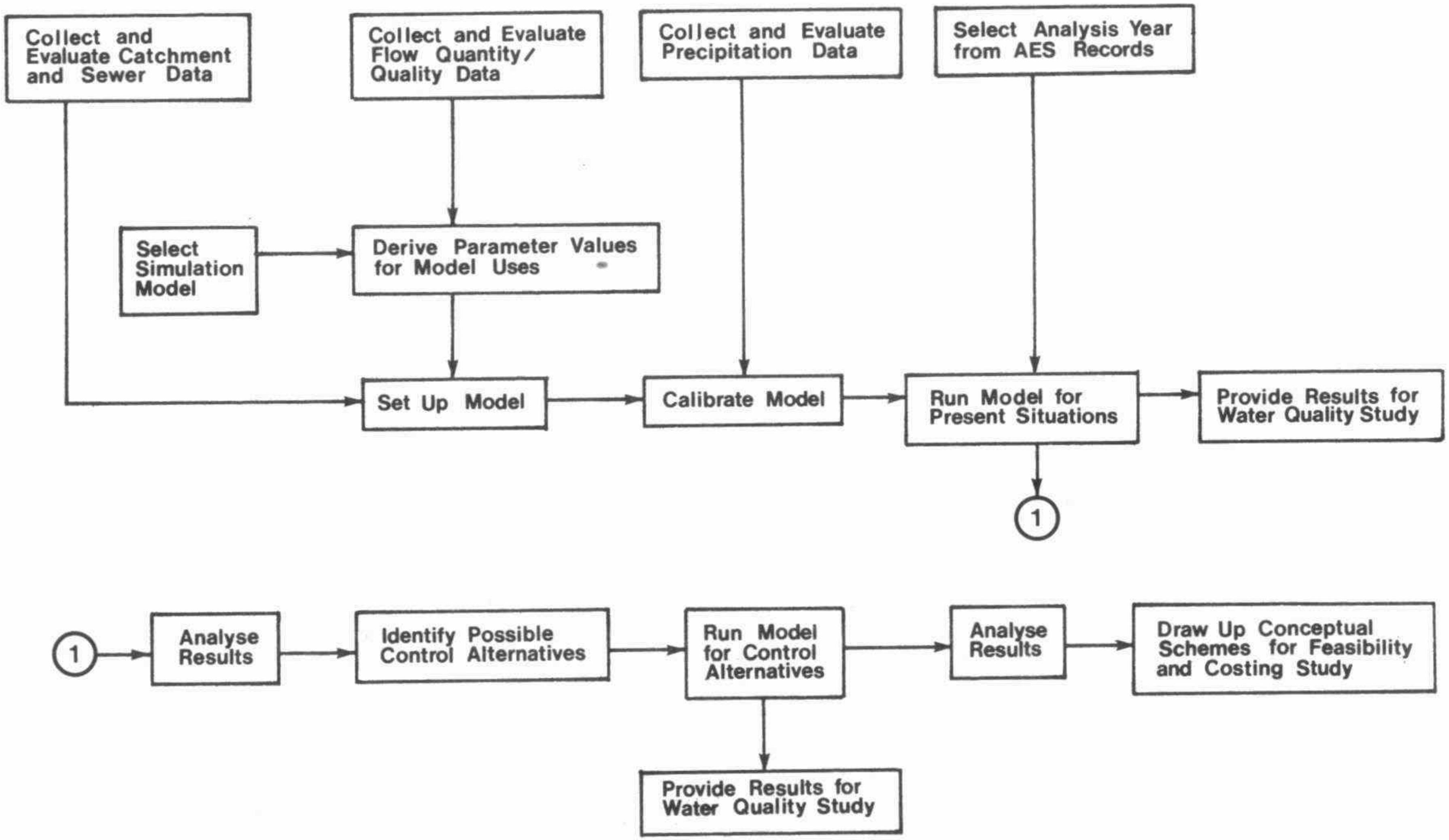


FIGURE 2.2 : STUDY ACTION PLAN

3.0 THE CATCHMENTS AND SEWERS

3.1 Description of Study Area

The study area (Figure 2.1) includes partly or wholly 4 cities of Metro Toronto, namely Etobicoke, North York, Toronto and York. Three areas in the study area are served mainly by combined sewers. They are marked 6.2.05 (Hillary Ave), 6.2.15 (Mt. Dennis) and 6.2.18 (Rockcliffe). Their total area is 1,331 ha of which 1,246 ha are combined sewer area and the rest is separate sewer area. The areas are connected to the Humber Sanitary Trunk Sewer (STS) which flows to the Humber WPCP. The remainder of the study area is served by separate sanitary and stormwater sewers. The sanitary sewer area has an area of 17,418 ha and is also connected to the Humber STS and the Humber WPCP. It was included in the study so that the total hydraulic loading on the Humber STS and the WPCP can be determined in the case of CSO control measures affecting these facilities.

The study area has a gentle terrain sloping southward in the general direction of the Humber river. The area in the lower reach of the Black Creek, where the combined sewer area exists, dips in a southwesterly direction as the Black Creek turns its way to join the Humber river.

About half of the combined sewer area is pre-World War II development. The other half was developed or redeveloped in the 1960's or early 1970's. Generally, building lots have short frontage. Stormwater runoff from roads, sidewalks and driveways is collected by gutters and underground sewers. Grassed swales or curb-side turf plots are uncommon. About two-thirds of the houses have their roof leaders connected to sewers. The other roof leaders discharge to ground surfaces. Streets are mostly smooth and well maintained. The top layer of soil in unpaved areas is mainly silty or clayey.

The sanitary sewer area is mainly an established sub-urban development. Its topography and surface characteristics were not of direct interest to this study because its stormwater runoff is not collected by the study sewer system. Wet-weather inflow/infiltration

(I/I) to the sanitary sewers, however, was accounted for as explained in Section 4.6.

3.2 Catchment Data

Most catchment data used were abstracted from a TAWMS data report (Gartner Lee Associates, 1983). The study area was divided into 3 main combined sewer catchments and 17 sanitary sewer catchments in anticipation of data needs for model simulation. The data are summarized in Table 3.1, several points of which are worth noting. The combined sewer area is only 7% of the whole study area. Its population density is more than double the average of the study area. Its wastewater production rate per person is only about half of the average of the sanitary sewer area's. It has a smaller percentage of industrial land use (Table 3.2) but a much larger component of low/medium residential use and commercial use.

In summary, the development characteristics of the combined sewer area are notably different from those of the sanitary sewer area, possibly because the combined sewer area is generally a few decades older (Gardner Lee Associates, 1983).

For model simulation purposes, the combined sewer catchments were further divided into subcatchments and data were compiled for the size of catchment or subcatchment, length of overland flow path, ground slope, length of street curb, percentage of impervious surface, population density and wastewater production rate for each land use. Similarly, data were compiled for each sanitary sewer catchment and the data included catchment size, population density and wastewater production rate for each land use.

An anomaly in the wastewater production rates was found in the original data (Gartner Lee Associates, 1983), but it had been corrected in the data presented above. The anomaly was that the sum of wastewater quantities calculated from the data was much larger than the average dry-weather flow (DWF) recorded at the Humber WPCP. In principle, the DWF should be the larger because it is the total of wastewater produced and groundwater infiltrated into the sewers. To correct the anomaly, additional water consumption data

TABLE 3.1

SUMMARY OF CATCHMENT DATA

	Area (ha)	Area (% of Study Area)	Population	Population Density (No./ha)	Wastewater Production Rate (2) m^3/d	m^3/p/d	m^3/ha/d
(A) Combined Sewer Area (1)							
Hillary	951.1	5.1	61,716	65	16,027	0.260	16.9
Mt. Dennis	183.5	1.0	9,564	52	3,237	0.339	17.6
Rockcliffe	196.1	1.1	9,994	51	2,553	0.256	13.0
Subtotal of (A)	1,330.7 (1)	7.1	81,274	61	21,817	0.268	16.4
(B) Black Creek San. Area	4,027.9	21.5	158,981	39	64,003	0.403	15.9
(C) Remaining San. Area	13,390.2	71.4	289,950	22	185,116	0.638	13.8
(D) All Sanitary Area (i.e. B + C)	17,418.1	92.9	448,931	26	249,119	0.555	14.3
(E) Study Area (i.e. A + D)	18,748.8	100.0	530,205	28	(3) 270,936	0.511	14.5

Notes:

(1) Including sanitary area of 84.7 ha.
(2) Including residential, commercial and industrial uses.
(3) Not including 17,280 m^3/d sewage from Mississauga.
Total including Mississauga was 288,216 m^3/d.

TABLE 3.2

LAND USE DISTRIBUTION (1)

	Residential Low/medium Density	Residential High Density	Commercial	Industrial	"Others" (2)
(A) Combined Sewer Area (Hillary + Mt. D. + Rock.)	71%	2%	7%	11%	9%
(B) Black Creek San. Area	43%	6%	3%	20%	28%
(C) Remaining San. Area	39%	3%	4%	26%	28%
(D) All Sanitary Area (i.e. B + C)	40%	4%	4%	24%	28%
(E) Study Area (ie. A + D)	42%	4%	4%	24%	26%

Note:

(1) Expressed in per cent as land use area divided by catchment area.
(2) Includes institutional uses and open spaces.

were obtained from a municipal water accounts department and the wastewater quantities were recalculated as explained in Appendix A1. The revised data compared well with observed DWF as explained in Section 4.2 and were adopted.

3.3 Description of the Combined Sewer System

A layout of the trunk combined sewers and the locations of the major overflow regulators is shown in Figure 3.1. Combined sewage flows to the Humber WPCP via the Black Creek STS, the Rockcliffe STS and the Humber STS. In wet weather, combined sewage up to 2.43 m^3/s (5.8 times of the DWF of the combined sewer area) is allowed to flow to the WPCP. Flow in excess of this limit is partly diverted to an existing detention tank on Hyde Avenue and the remainder overflows to the Black Creek at the Site 3, Mt. Dennis and Rockcliffe overflow regulators. The Hyde Avenue tank has a capacity of 7,800 m^3. When full, the tank also overflows to the Black Creek. The detained sewage is returned to the WPCP via the Black Creek STS after a storm.

The tank is an underground concrete tank with an annexed operating room on the ground. Normally, the valve of the underdrain for return flow is open and the tank is empty at the onset of a storm. When inflow first arrives at the tank, the rising water level closes the valve automatically. After a storm, the valve is opened manually to drain the tank. The draining of a full tank takes about 11 hours.

Only the Hillary catchment diverts excessive flow to the tank. The catchment's flow to the WPCP in both dry and wet weather follows the dashed lines. In wet weather, excessive flow diverted to the tank follows the dotted lines. The diversion is controlled by regulators other than those mentioned earlier. Since the latter set of regulators does not divert any combined sewage to receiving waters directly, these regulators are of no direct interest to this report and will not be mentioned further.

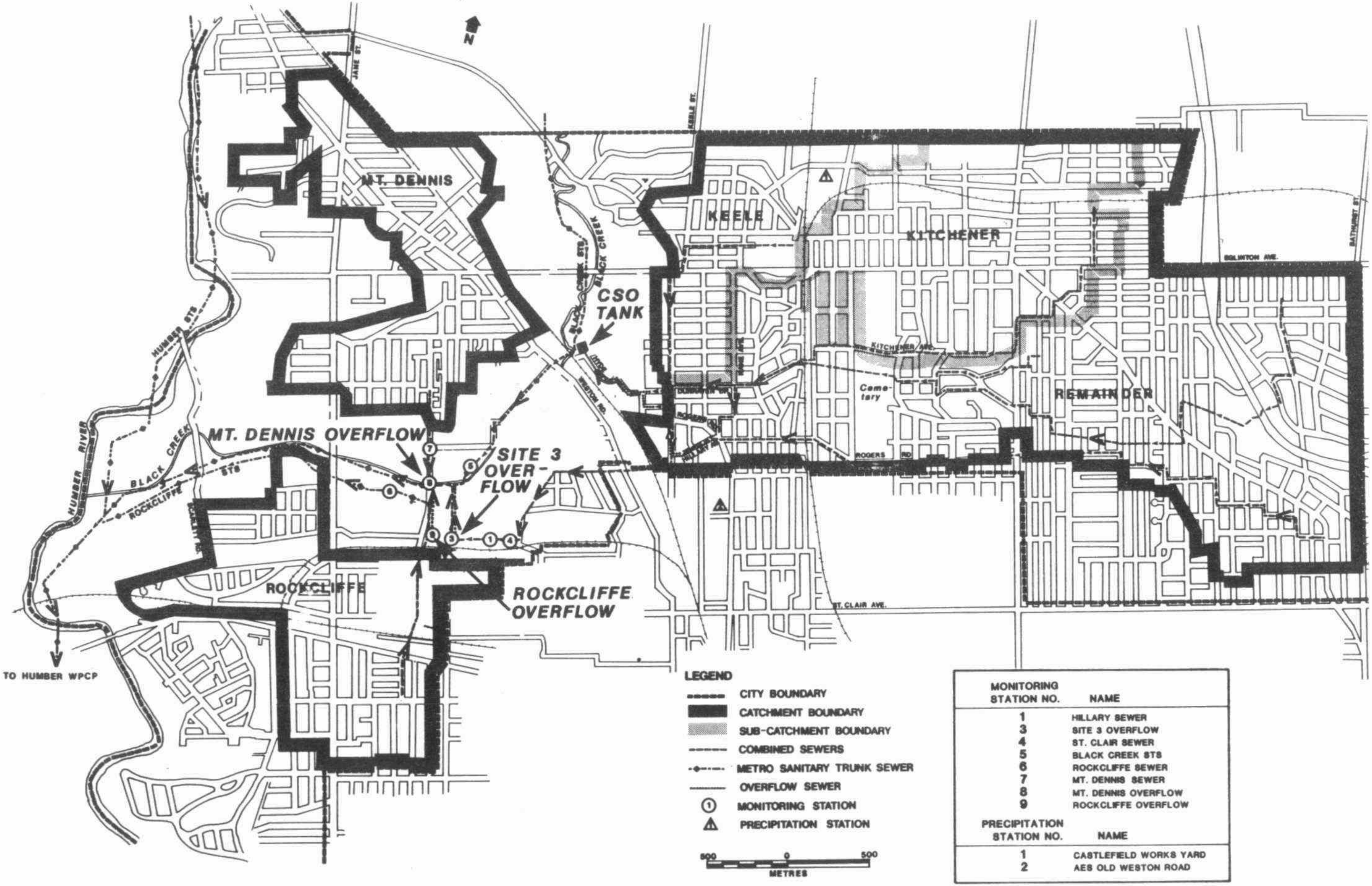


FIGURE 3.1 : MAP OF COMBINED SEWER AREA

Overflow to the Black Creek at regulators will occur when flow in the sewers at the regulator points reach the following rates (threshold capacities) determined from sewer engineering details:

Regulator	Threshold Capacity (m^3/s)	DWF at Regulator (m^3/s)
Site 3	1.64	0.312
Mt. Dennis	0.32	0.060
Rockcliffe	0.47	0.050

3.4 The Berry Road Regulator and the Humber WPCP

The Berry Road regulator is installed in the Humber STS at Berry Road (Figure 2.1). It discharges to the Humber river. It is the last regulator to limit the flow to the Humber WPCP.

This regulator may overflow in one or both of two circumstances which are illustrated in Figure 3.2. Overflow will occur if the flow at the upstream end of the regulator exceeds the threshold capacity of $10.2 m^3/s$. Overflow will also occur if the operator of the Humber WPCP closes some of the gate valves of the WPCP to restrict the flow entering the WPCP. In this circumstance, the restricted flow will back up in the Humber STS and overflow at the regulator.

It was assumed in this study that the allowable peak flow to the WPCP was equal to the peak primary treatment capacity of 11.8 m^3/s and that the primary effluent in excess of the secondary treatment capacity of 9.6 m^3/s bypassed the secondary treatment process and was discharged to the effluent outfall, after chlorination. The peak capacity of the effluent outfall is 11.8 m^3/s. The data, illustrated in Figure 3.3, were supplied by the Metro Toronto authority. The design peak primary and peak secondary treatment capacities were defined as the maximum capacities above which effluent qualities begin to deteriorate. (Metro Toronto, 1984).

In October 1985, after the draft of this report was prepared, the Metro Toronto authority revised the peak primary treatment capacity from 11.8 m^3/s to 7.8 m^3/s (if primary treatment is not followed

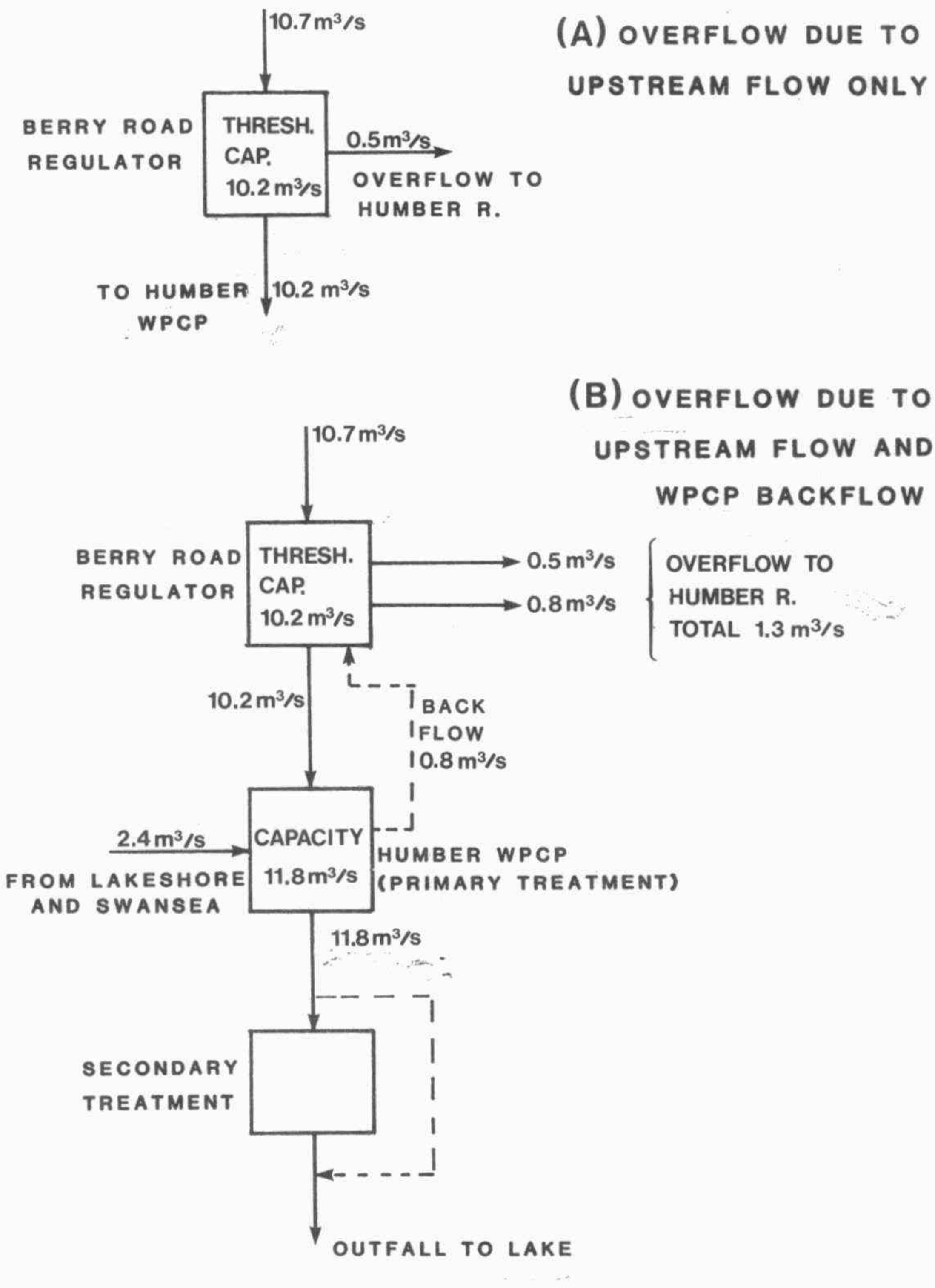
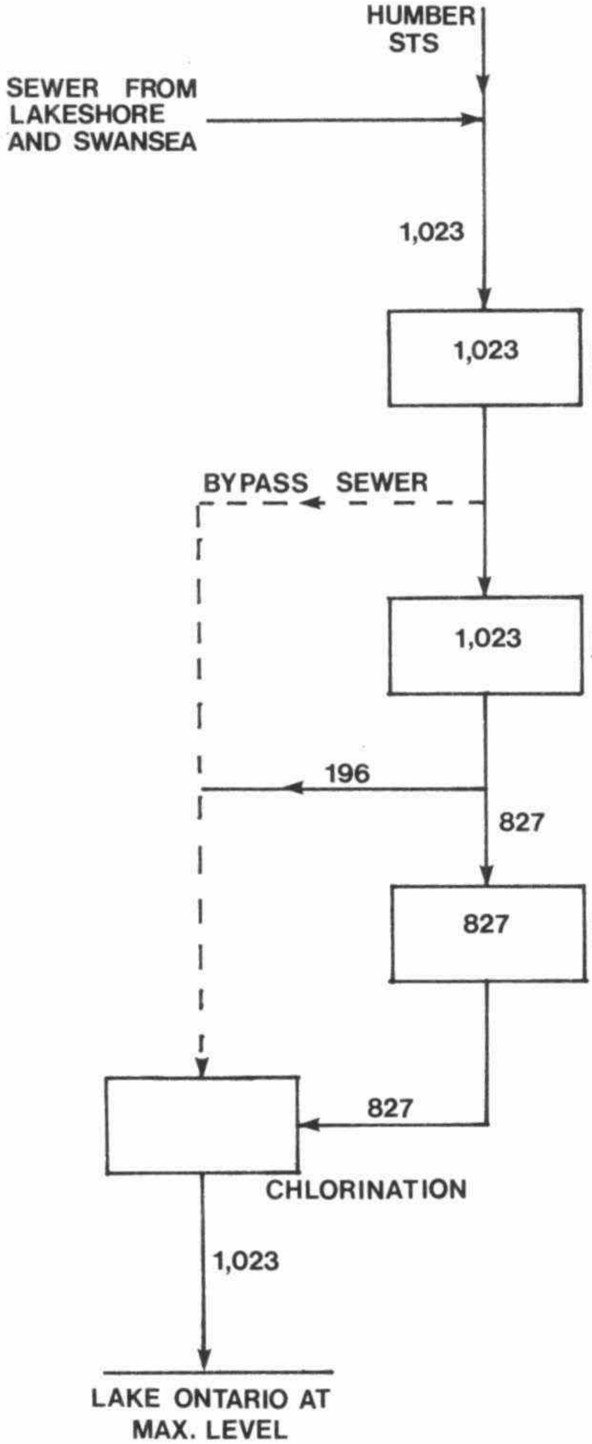


FIGURE 3.2: CAUSES OF OVERFLOW AT BERRY ROAD REGULATOR



NOTES:

(1) ALL VALUES ARE IN 1000 m^3/d.

(2) AVG DESIGN DWF IS 408 x 10^3 m^3/d.

(3) DIAGRAM ABRIDGED FROM METRO TORONTO DRAWING.

CONVERSION TABLE

10^3 m^3/d	m^3/s
196	2.3
408	4.7
827	9.6
1,023	11.8

FIGURE 3.3: DESIGN PEAK SEWAGE FLOW OF HUMBER WPCP

by secondary treatment) and both the primary and secondary treatment capacities to 8.9 m^3/s (if primary treatment is followed by secondary treatment). The Metro Toronto authority indicated that the revision was aimed at maintaining the effluent quality at all time. The revision would render CSO Control Scheme 2 (Section 7.3) infeasible, but no other conclusion in the report would be nullified.

An anomaly in the revision is observed in that the Humber WPCP would have a smaller throughput capacity if the WPCP was operated as a primary treatment plant than if it was operated as a secondary treatment plant. Since both the original and the revised capacities are hypothetical values and they conflict with each other, an in-WPCP evaluation is recommended if CSO Control Scheme 2 is to be considered for adoption. In the meantime, the original capacities were assumed in the study.

3.5 Sewer Data

Numerical data needed for computational analysis were mostly abstracted from the TAWMS data report (Gartner Lee Associates, 1983). The data included sewer sizes and invert elevations.

3.6 Minor Combined Sewer Catchments

Two minor combined sewer areas exist in catchments No. 5.1.19 (Keele Street/St. Clair Avenue) and 5.1.21 (Ardagh Street/Jane Street) shown in Figure 2.1. These two areas are outside the Hillary, Mt. Dennis and Rockcliffe catchments. As the two areas together are only 13.4 ha, they were regarded as minor local cases and were ignored in the study.

4.0 FLOW AND PRECIPITATION DATA

4.1 Data Collection Program

Dry-weather flow (DWF) and wet-weather flow (WWF) were monitored and sampled at selected locations of the sewer network for purposes shown in Table 4.1. The locations of the field stations are indicated in Figure 3.1.

Data collection efforts for the combined sewer area were concentrated on the Hillary catchment which makes up 72% of the combined sewer area. Four stations, namely, the inlet and the overflow weir of the Hyde Avenue tank and Sites 1 and 3 of the Hillary sewer, were needed to completely define the combined sewage flow of the catchment. The DWF at Site 1 was the total DWF of the catchment.

The Mt. Dennis and Rockcliffe stations were monitored to provide supplementary DWF quality data and CSO information.

Station 5 at the outlet of the Black Creek sanitary sewer area was used to provide data representative of DWF qualities of the entire sanitary sewer area of the study. Additionally, the data from this station, together with daily flow data obtained from the Humber WPCP log sheets, were used to determine the DWF quantities as well as dry and wet-weather infiltration rates of the sanitary sewer area.

The St. Clair Avenue sanitary sewer serves a catchment of only 47 ha. It was monitored and sampled because it receives flow from a number of meat packaging and protein recovery plants.

Precipitation was gauged at the Castlefield Works Yard station.

TABLE 4.1

FIELD DATA COLLECTION PROGRAM

Location of Station	Station I.D. No.	Type of Data Collected: DWF Quan.	DWF Qual.	WWF Quan.	WWF Qual.	Use of Data
Castlefield Works Yard						Precipitation data to correlate with observed combined flow.
Hyde Ave Tank Inlet				X		Part of Hillary WWF; tank utilization; model calibration.
Hyde Ave. Tank Overflow	2			X		As above; overflow information.
Site 3 Regulator	3			X		Part of Hillary WWF;overflow information.
Hillary Sewer (After Site 3)	1	X	X	X	X	Part of Hillary WWF; total DWF of Hillary.
Mt. Dennis Sewer	7	X				Supplementary DWF quality information of combined sewer area.
Rockcliffe Sewer	6	X				As above.
Mt. Dennis Regulator	8			X		Check of overflow events.
Rockcliffe Regulator	9			X		As above.
Black Creek San. Sewer Area	5	X	X			Representative DWF information of sanitary area.
St. Clair Ave. Sewer	4	X	X			Special industrial wastewater.

Note: DWF = Dry-weather flow
WWF = Wet-weather flow

Most field data were collected in the period between April and October, 1983, although some DWF data were obtained in the fall of 1982. A summary of the instrumentation and data collection protocol is in Appendix B1.

All samples were analyzed by the Ministry of the Environment laboratory in Rexdale.

4.2 DWF Quantities

In calculating DWF, a dry-weather day was defined as a day without precipitation in the preceding 24 hours.

DWF data of 27 days were collected from Site 1 and analyzed to obtain the daily average flow, the ratios of flow variations in the days of the week and the ratios of flow variations in the hours of the day. These ratios were useful for synthesizing the DWF baseline of a combined sewage hydrograph whose DWF baseline could not be measured directly on a wet day. The observed average DWF at Site 1 was 26,946 m^3/d or 0.437 m^3/person/d. Detailed data are in Appendix B2.

Similarly, 14 days of observed DWF data were collected at Site 5. The average DWF was 86,618 m^3/d or 0.545 m^3/person/d. Detailed data are in Appendix B2.

The Humber WPCP data are summarized in Table 4.2. The data marked with an asterisk were considered as outliers because their values were incongruously smaller than the average of the remaining data set. The average DWF at the WPCP was 373,800 m^3/d which included the DWF of 17,280 m^3/d from the City of Mississauga discharged to the North Mimico Creek STS. Excluding the Mississauga flow, the average DWF at the WPCP was 356,520 m^3/d or 0.672 m^3/person/d. It is clear that CSO control may consider utilizing spare primary treatment capacity but not spare secondary treatment capacity.

TABLE 4.2

RECORDED HUMBER WPCP DRY WEATHER FLOW

	1980		1981		1982		1983		4-Year
	No. of Dry Days	Avg Daily DWF	No. of Dry Days	Avg Daily DWF	No. of Dry Days	Avg Daily DWF	No. of Dry Days	Avg Daily DWF	Avg Daily DWF#
January	22	348.1	28	261.9*	18	336.8			342.4
February	24	326.5	10	259.8*	18	334.4			330.5
March	16	320.7	24	246.1*					320.7
April	8	350.9	17	252.3*	18	392.4			371.7
May	20	333.6	13	216.6*	16	364.6	6	371.8	356.7
June	11	408.8	14	239.9*	12	415.3	18	362.3	395.5
July	12	359.0	16	401.2	24	338.7	23	325.9	356.2
August	20	391.1	13	411.6	13	309.7	14	372.2	371.1
September	17	427.1	10	380.3			18	352.6	386.7
October	11	456.4	10	356.2			14	342.2	384.9
November	16	367.8	23	383.0					375.4
December	19	363.5	16	331.5					347.5

For Season of April - October In 1980 - 1983#

Average Daily DWF 373.8 x 10^3m^3/d (4.33m^3/s)

Standard Deviation 36.1 x 10^3m^3/d

Notes:

(1) All volumes in 10^3m^3/d; unless stated otherwise.
(2) Data taken from WPCP operation records.
(3) * Data not used.
(4) # Excluding data marked with *.

There are two similarities between the observed DWF (Table 4.3) and the wastewater production rates (Table 3.1) mentioned in Section 3.2. One similarity is that the observed DWF of the combined sewer area (0.447 m^3/person/d) is about half of that of the sanitary sewer area (0.713 m^3/person/d) as the wastewater production rate of the combined sewer area (0.268 m^3/person/d) is half of that of the sanitary sewer area (0.555 m^3/person/d). The other similarity is that the observed DWF of the Black Creek sanitary sewer area and of the remaining sanitary sewer area are in the ratio of 1.0 to 1.5 (i.e. 0.545 m^3/person/d to 0.806 m^3/person/d) as the wastewater production rates of the same two areas are in the ratio of 1.0 to 1.6 (i.e. 0.403 m^3/person/d to 0.638 m^3/person/d). The similarities substantiate the reasonableness of the revised wastewater production data.

The dry-weather infiltration (Table 4.3) was 40.5% of the DWF in the combined sewers and ranged from 20.8% to 26.1% in the sanitary sewers. The higher percentage for the combined sewers was probably due to the combined sewers being older. However, 40.5% is not excessively high, because it is equivalent to 0.179 m^3/person/d only and is well within the range from 0.212 m^3/person/d to 0.593 m^3/person/d of dry-weather infiltration recommended for sewer design (Ministry of Environment, 1984).

4.3 DWF Qualities

DWF sample results are shown in Table 4.4. Details are in Appendix B2. The cadmium results shown in the Table were taken from the supplementary sampling in 1983 at the Mt. Dennis and Rockcliffe sewers, because samples collected in the Hillary catchment in 1982 were not analyzed for cadmium.

TABLE 4.3

COMPARISON OF DWF QUANTITIES

Catchment	Area (ha)	Population	Wastewater Production (m^3/d)	Observed Dry Weather Flow (DWF): Total DWF (m^3/d)	m^3/person/d	m^3/ha/d	Dry Weather Infiltration (2)
(A) Combined Sewer Area							
Hillary	951.1	61,716	16,027	26,946	0.437	28.3	40.5%
Mt. Dennis	183.5	9,564	3,237	5,184 (4)	0.542	28.3	40.5% (3)
Rockcliffe	196.1	9,994	2,553	4,320 (4)	0.432	22.0	40.5% (3)
Sub-Total of (A)	1,330.7 (1)	81,274	21,817	36,288	0.447	27.3	40.5%
(B) Black Creek San. Area	4,027.9	158,981	64,003	86,618	0.545	21.5	26.1%
(C) Remaining San. Area	13,390.2	289,950	185,116	233,614 (6)	0.806	17.4	20.8%
(D) All Sanitary Area (i.e. B + C)	17,418.1	448,931	249,119	320,232	0.713	18.4	22.2%
(E) Study Area (i.e. A+D)	18,748.8	530,205	270,936	356,520 (5)	0.672	19.0	24.0%

Notes:
(1) Including sanitary area of 84.7 ha.
(2) Expressed as (Obs. DWF-TWP)/Obs. DWF.
(3) Assumed same as Hillary.
(4) Derived value. Calculated as TWP/(1-.405).
(5) WPCP record of 373,800 m^3/d less 17,280 m^3/d from Mississauga.
(6) Derived value. Calculated as 356,520 - 36,288 - 86,618.

TABLE 4.4

DWF QUALITIES

Hillary Catchment Station No. 1	BOD5	RSP	PPUT	PPO4	CUUT	PBUT	ZNUT	CDUT (1)
Avg. DWF(m^3/s) 0.312								
Avg. Conc.(mg/l)	208.9	245.4	5.42	2.42	0.16	0.06	0.24	.014
No. of Samples	11	11	11	11	11	11	11	12
Std. Dev. (mg/l)	75.1	92.4	2.66	0.83	0.11	0.03	0.10	.007
Black Creek San. Area Station No. 5								
Avg. DWF(m^3/s) 1.000								
Avg. Conc.(mg/l)	191.9	173.8	5.66	2.58	0.36	0.04	0.17	
No. of Samples	14	14	14	14	14	14	13	
Std. Dev. (mg/l)	52.4	44.3	2.94	0.89	0.13	0.02	0.05	
St. Clair Ave. Sewer Station No. 4								
Avg. DWF(m^3/s) 0.017								
Avg. Conc.(mg/l)	2077	1067	29.2	19.4	0.07	0.029	0.28	
No. of Samples	9	9	8	9	9	9	9	
Std. Dev. (mg/l)	547	639	6.95	5.67	0.032	0.014	0.037	

Note:

(1) Data from Mt. Dennis and Rockcliffe Catchments

RSP = Suspended Solids
CUUT = Copper.
CDUT = Cadmium
ZNUT = Zinc.
PBUT = Lead.
PPO4 = Filtered Reactive Phosphorus.
PPUT = Total Phosphorus.

The results of the Hillary catchment suggest that the DWF of the combined sewer area was quite typical of municipal sanitary wastewaters. The standard deviation values suggest that the qualities were reasonably stable.

The results of the Black Creek STS were generally comparable to those of the combined sewer area, although the suspended solids, lead and zinc concentrations were somewhat lower and copper concentration was higher than those of the combined sewer area. No explanation can be given for the differences. The results of the Black Creek sanitary sewer area were assumed to be representative of the entire sanitary sewer area in later model simulation work.

As expected, the DWF of the St. Clair Avenue sanitary sewer did show high concentrations of BOD_5, suspended solids, total and soluble phosphorus that are normally observed in meat prosessing wastewater. The flow from this sewer, however, is only 1/78 of the flow of the Black Creek STS and, therefore, was not of particular importance to this study.

4.4 Precipitation Data

Precipitation data obtained at the Castlefield Works Yard station in April to October, 1983 were segregated into events for further analysis. A storm event was defined as a period of precipitation not separated by a no-precipitation period of longer than 6 hours. The use of the 6-hour period for separating events was arbitrary but it was believed that this duration was reasonable because the impact of a storm on the sewer system was expected to have diminished to a negligible degree after precipitation had ceased for 6 hours. The same period was used for defining storm events in another TAWMS project (Pitt, 1984).

Precipitation results are shown in Table 4.5. There were 58 events in the season. The total seasonal precipitation was 369.8 mm and the average precipitation was 6.4 mm per event. The season in 1983

TABLE 4.5

OBSERVED PRECIPITATION AT CASTLEFIELD WORKS YARD

(A) Summary

Summary	Observed (1983)	AES 1966-81 Avg.
Observation Period:	April-October	April-October
Total No. of Events:	58	64
Total Precipitation:	369.8 mm	447.7 mm
Avg. Precipitation per Event:	6.4 mm	6.9 mm

(B) Frequency Distribution of Observed Events

	Event Precipitation (mm), Not More Than							More Than
	2	4	6	8	10	15	20	20
No. of Events	21	11	6	6	2	5	3	4
Cumulative No. of Events	21	32	38	44	46	51	54	58

Note: AES = Atmospheric Environmental Services of Environment Canada.

was somewhat drier compared with the statistics of the average season between 1966 and 1981 of the Environment Canada precipitation station at Old Weston Road.

4.5 Combined Sewage Quantities and Qualities

Combined sewage quantity data were collected in 14 storm events in the Hillary catchment. This section provides a general appraisal of the data. The use of the data for model simulation will be discussed in the appropriate sections later.

Precipitation in the storms ranged from 4 mm to 30 mm (Table 4.6). Inflow to the Hyde Avenue tank occurred in each of the 14 events but the tank was filled only in 9 events, most of which had a precipitation more than 12 mm. Site 3 overflowed in each of the 14 events. Hydrographs of these events are in Appendix B2 and an example is shown in Figure 4.1. One obvious conclusion from the example is that reduction of DWF will not be effective in reducing CSO since the DWF rate is only a small fraction of the combined sewage flow rate. On the other hand, reduction in stormwater runoff, if it is feasible, will effectively reduce the combined sewage flow rate.

Observed combined sewage suspended solids (SS) concentrations are plotted against observed flow rates in Figure 4.2. Detailed data are in Appendix B2. SS are of particular interest because the concentrations of several of the other pollutants could be related to SS concentrations as will be demonstrated later.

The SS data appear reasonably normal except possibly in two places. First, the data of the beginning part of the event on August 8, 1983 were erroneous because the Black Creek momentarily flooded the overflow weir of the Hyde Avenue tank. The erroneous data points were not used. Second, there was a sudden change from a low SS concentration to a high SS concentration in the last two points on the curve of the event on September 16, 1983. A trend like this was not observed in the other events. The last point on the curve was

TABLE 4.6

LIST OF MONITORED EVENTS IN HILLARY CATCHMENT

Date	Event Precip. (mm)	Use of Data: Model Calib.	Use of Data: Model Verif.	Use of Data: Load Rate	Occurrence of Phenomenon: Inflow to Tank	Occurrence of Phenomenon: Overflow From Tank	Occurrence of Phenomenon: Overflow at Site 3	Comments
30 Apr 83	19.3	Y			Y	Y	Y	
1 May 83	7.9	Y			Y		Y	
2 May 83	14.0	Y			Y	Y	Y	
19 May 83	21.9	Y		Y	Y	Y	Y	
22 May 83	11.6	Y			Y		Y	
29 May 83	11.6		Y	Y	Y		Y	
6 June 83	6.7	Y		Y	Y		Y	
4 July 83	4.3		Y	Y	Y		Y	
8 Aug 83	24.7			Y	Y	Y	Y	Part of Event.
22 Aug 83	20.2		Y	Y	Y	Y	Y	
16 Sep 83	29.5			Y	Y	Y	Y	Flow data incomplete.
21 Sep 83	7.4	Y			Y	Y	Y	
12 Oct 83	19.0			Y	Y	Y	Y	Precip. gauge problem. No SS data.
13 Oct 83	14.2	Y		Y	Y	Y	Y	

Note: Y = Yes

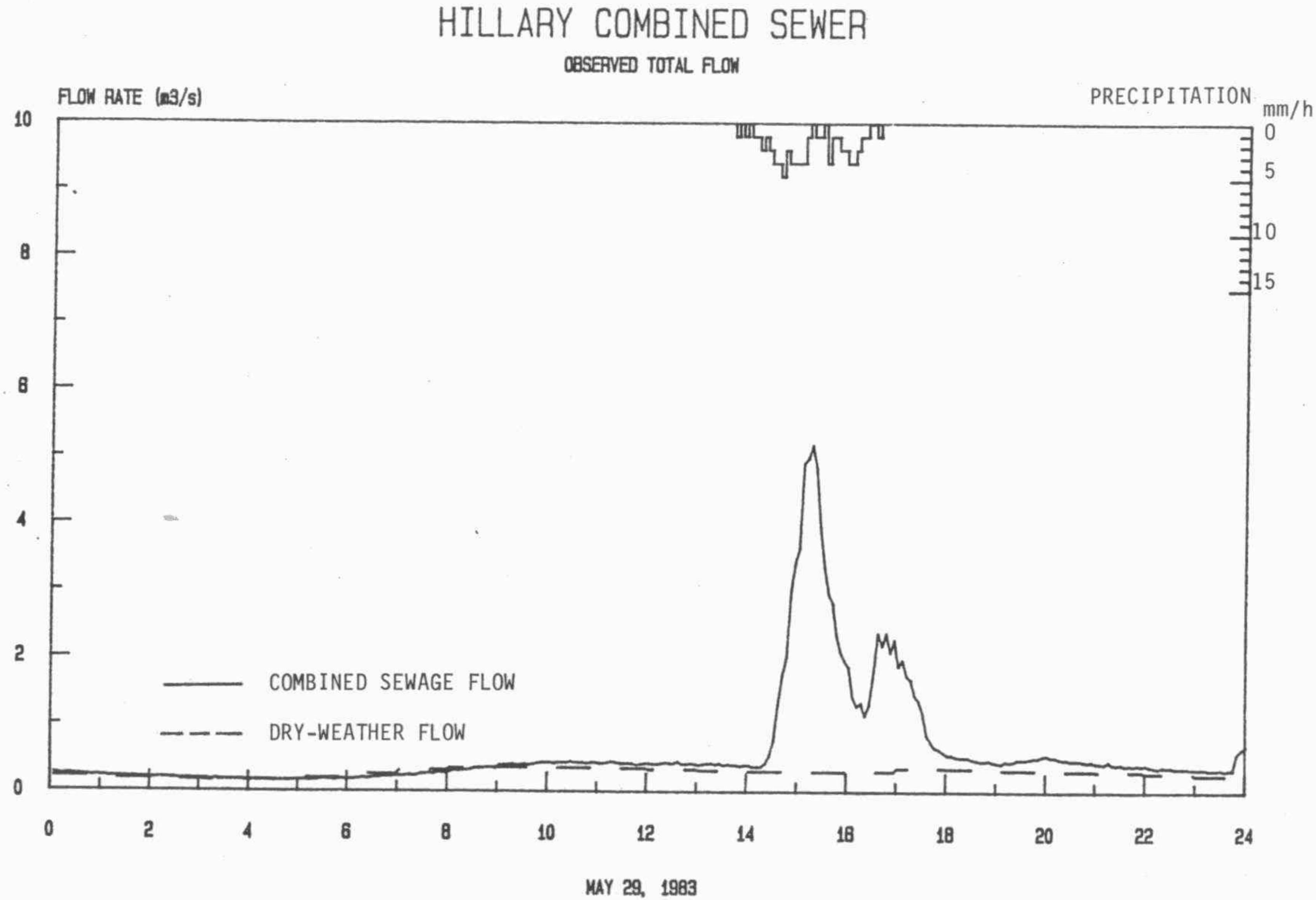


FIGURE 4.1 : COMBINED SEWAGE HYDROGRAPH

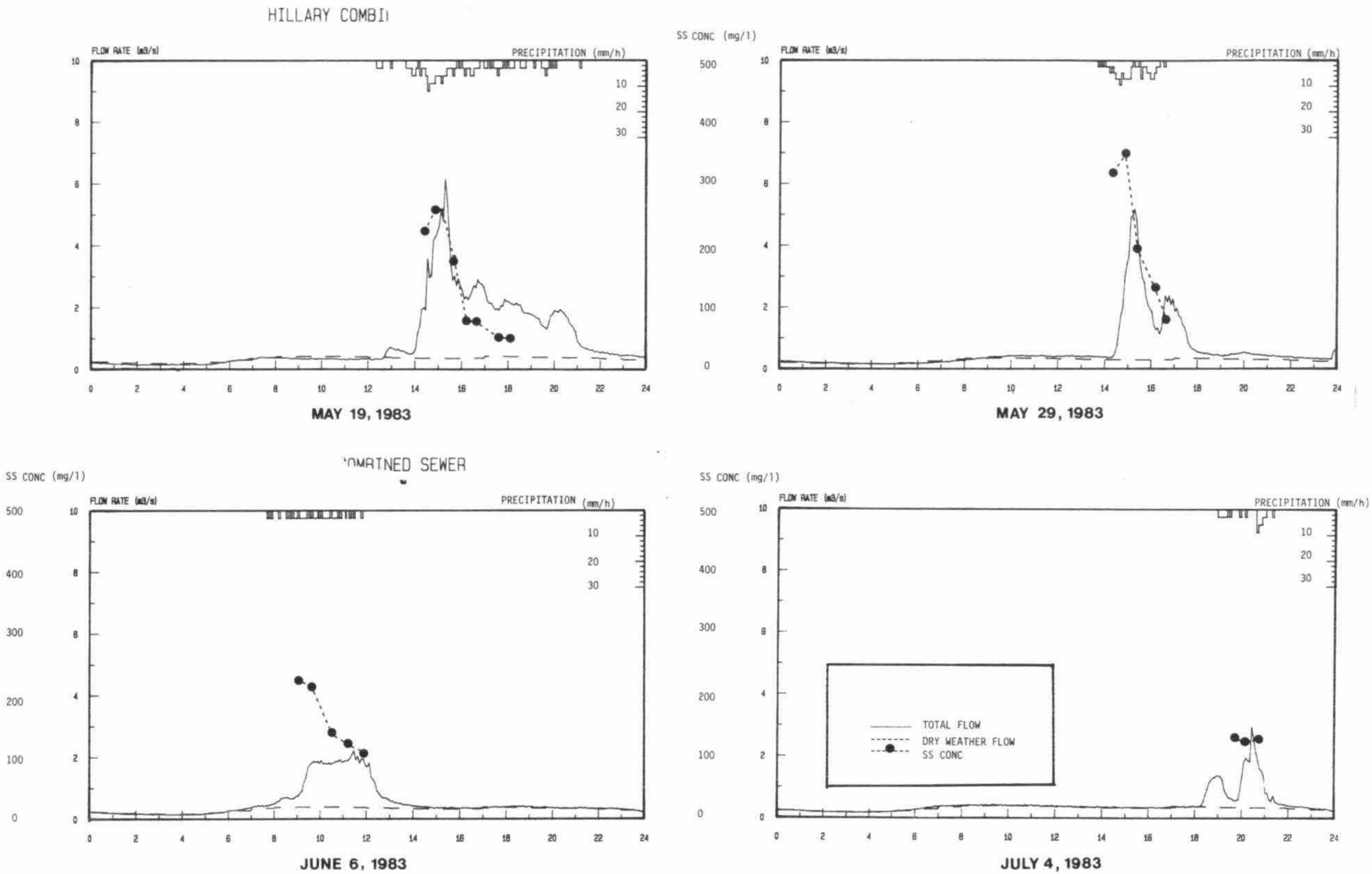


FIGURE 4.2 : COMBINED SEWAGE SUSPENDED SOLIDS CONCENTRATIONS (sheet 1 of 2)

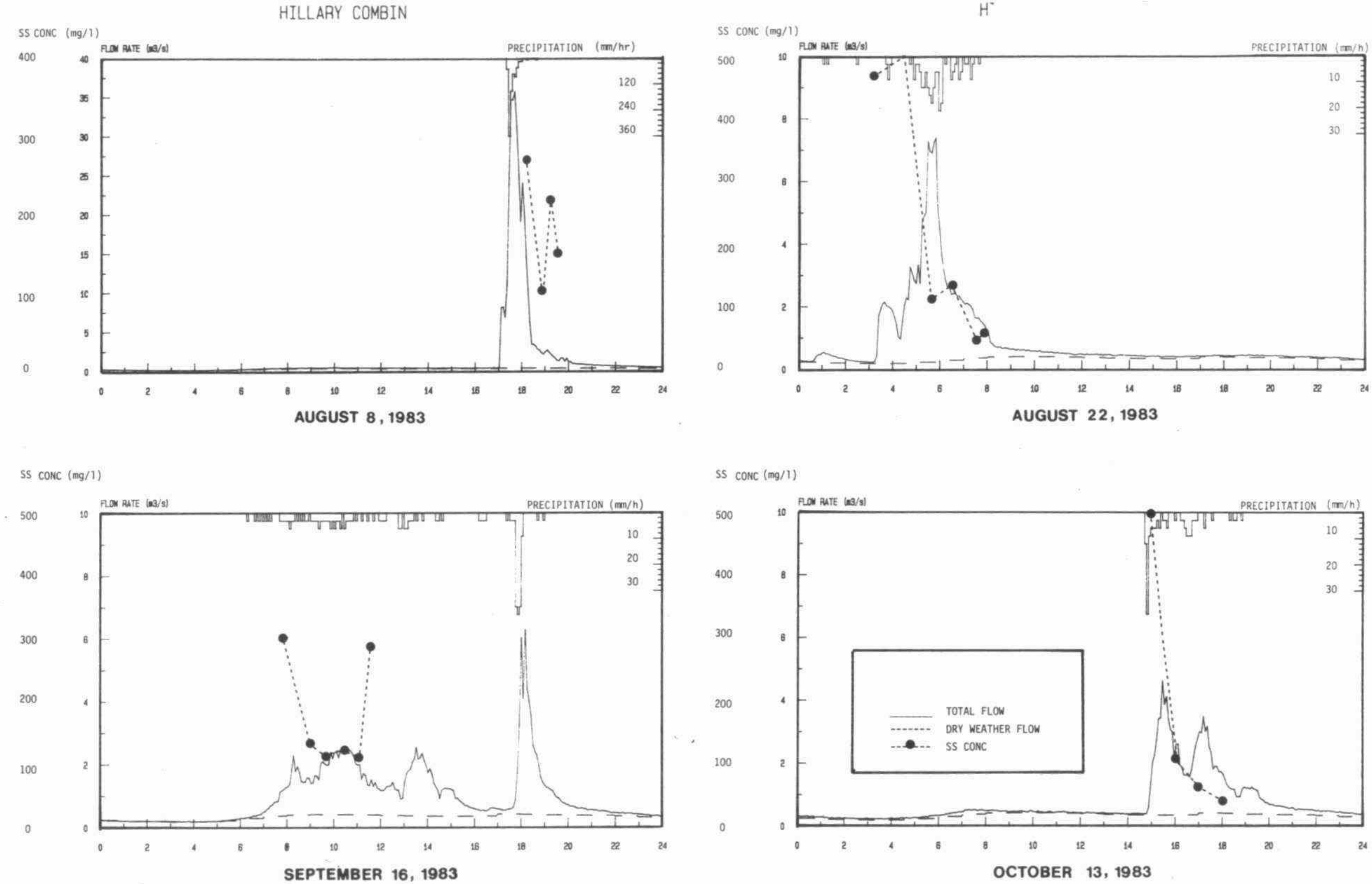


FIGURE 4.2 : COMBINED SEWAGE SUSPENDED SOLIDS CONCENTRATIONS (sheet 2 of 2)

apparently a random error which may be expected according to theories of statistics. The cause of the error could not be traced. Since this point lies well within reasonable bounds of the entire set of SS concentrations (the point is about 1 standard deviation from the mean concentration of the entire set), the point was not rejected.

It is interesting to note that the SS concentrations varied approximately proportionally with the flow rates. This relationship provided the basis for relating the SS concentration to the flow rate in the model simulation discussed later.

The flow-weighted concentrations of SS, BOD_5, total phosphorus and fecal coliforms of the observed events are summarized in Table 4.7. The results appear to be comparable to literature values as illustrated in Table 4.8. The comparison should not be made too strictly, of course, because combined sewage is variable in nature and the catchments are not identical.

4.6 Wet Weather Inflow/Infiltration of Sanitary Sewer Area

There were indications that the wet-weather inflow/infiltration (I/I) in the sanitary sewer area was notable in certain locations but negligible in some others (City of Etobicoke, 1983). However, no field studies had been carried out by municipalities to estimate the magnitudes and distribution of wet-weather I/I in the sanitary sewer area. Resources and time allotment for this CSO study precluded the undertaking of a field study for this purpose either.

In this study, the wet-weather I/I in the sanitarty sewer area was estimated from wet-weather daily flow volumes recorded on the Humber WPCP log sheets. Data of year 1980 were used since they gave the best correlation with the precipitation of the year. The following relationship was derived:

TABLE 4.7

SUMMARY OF OBSERVED COMBINED SEWAGE QUALITY DATA

	Number of Events Sampled	Total No. of Samples	Flow-weighted Avg. Conc. (mg/l)	Coef. of Variation
Suspended Solids	8	42	196.	0.68
BOD5	9	48	55.	0.61
Total Phosphorus	9	44	1.96	0.35
Filtered Phosphorus	9	48	0.44	0.73
Cadmium	7	38	0.006	0.83
Copper	7	38	0.119	0.59
Lead	7	38	0.182	0.61
Zinc	7	38	0.300	0.62
Fecal Coliforms	14	81	6.218*	0.11

* Log mean, No./100ml

TABLE 4.8

COMPARISON OF COMBINED SEWAGE CONCENTRATIONS

Location	Average Concentrations (mg/l) (1) Suspended Solids	BOD5	Phosphorus	F.Coliforms No./100 ml	Site Descriptions & Remarks
Hillary Catchment	196	55	1.96		Study site.
Belleville WPCP Overflow (2)					Mainly resid./com. Total city 2,350 ha. CSO area 50 ha.
Events with "1st Flush"	522	118	5.3	-	9 events with "1st flush"
No "1st Flush"	191	68	4.0		5 events no "flush"
New Haven CSO Area (3)	226	302	-	-	Residential. 6 ha. 2 events observed.
Milwaukee CSO Area (4)	361	87	-	Range 15-12,000	Area 2,428 ha. 70% resid.,18% com., 11% ind. 15 events for SS,12 for BOD, 3 for FC.
Cleveland CSO Area (5)	234	92	-	4.5x10^6	Area 24,800 ha. Population 600,000. Resid/com/ind. With some large industries. Data of 193 to 197 observations.

Notes: (1) Except Fecal Coliforms
(2) Kronis, H. , 1975
(3) Cermola, J.A. et al, 1979
(4) Meinholz, T.L. et al, 1979
(5) Nebolsine, R. et al, 1972

$$II = 1.475 \times P^{1.446}$$

where II = I/I in a storm event in 1,000 m^3

P = Precipitation in event in mm.

Coefficient of correlation is 0.66

This relationship was used later to predict the I/I volume of a given storm event in model simulation. The I/I volume was distributed over the storm event duration, using a unit hydrograph which was derived from the Humber WPCP hourly flow volumes in the summer of 1983. Figure 4.3 is an example of the I/I hydrograph so generated. Details of the methodology for computing the I/I are in Appendix B3.

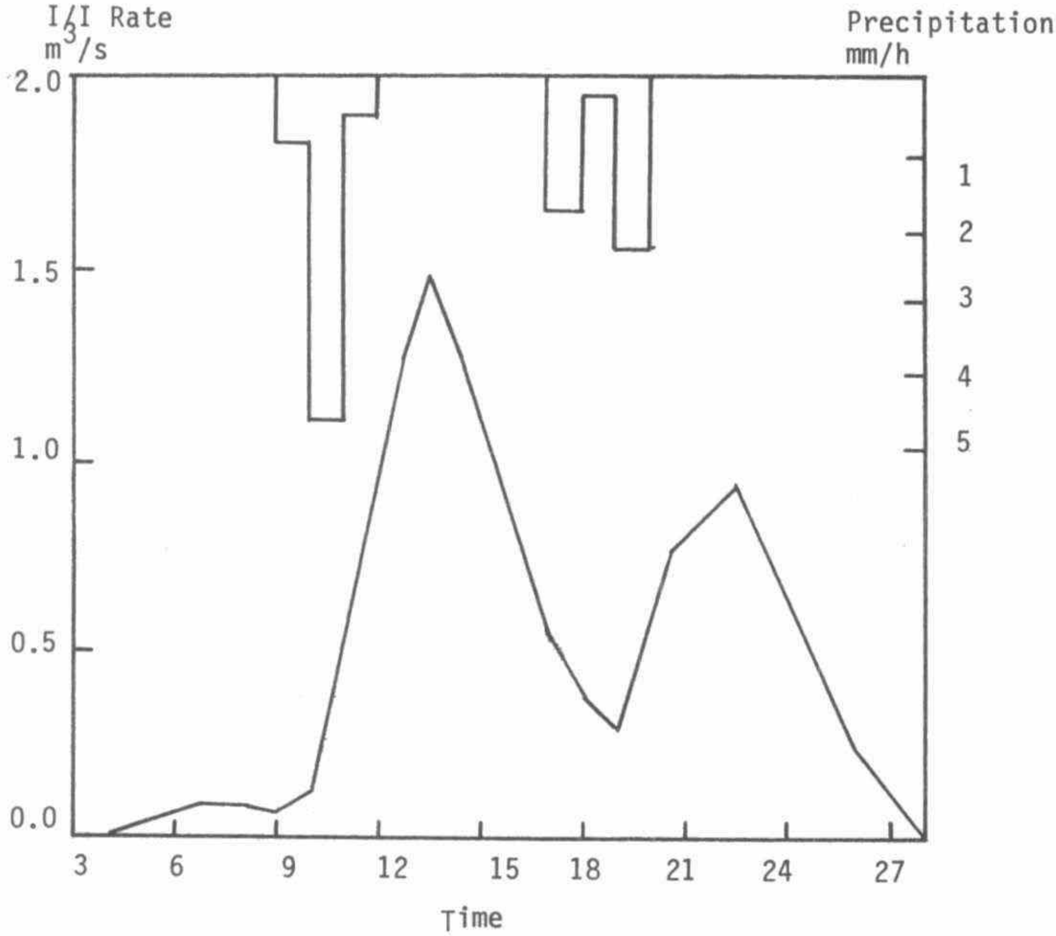


FIGURE 4.3: ILLUSTRATION OF INFLOW/INFILTRATION HYDROGRAPH (JULY 10, 1979)

5.0 MODEL SIMULATION

5.1 Need for Judicious Use of Model

Section 5.0 presents briefly a general concept of computer model simulation and describes the setting up of the model for this CSO study. Applications of the model will be discussed in Sections 6.0 and 7.0.

In general, it is not practical to obtain CSO statistics for a complete, typical hydrologic season by field studies alone. Nor is it possible to forecast from observed data the changes in the statistics resulted from CSO control. The missing information needs to be obtained by model simulation as was done in this study. The role of data collection presented earlier was to provide an understanding of the responses of the study system to hydrologic changes and to obtain data required for using the model.

Model simulation was already used in engineering practice in the age of hand calculation. The advent of the computer makes it possible to calculate much faster and to do complex mathematics with fewer simplifying assumptions. The computer also makes it possible to store and print voluminous information at great speed and low cost and this ability meets the needs of input/output requirements of model simulation just well. In these ways, the computer augments the capability of model simulation, but the computer is no substitute for the knowledge of and the experience in the application of engineering principles in model simulation. The need for applying computer model simulation with caution may be reflected by the following quotation:

> "However, in a major model, there are many relationships which are incorporated without being unequivocably assessed to be true; this important aspect seems to have escaped major criticism..... computer approaches not based upon proved scientific relationships can be highly misleading." (Whipple, 1977).

Even if the theories used in a model are sound and the modelling is done properly, the need for intelligent appraisal of modelling results cannot be over-emphasized. This is true of any urban environmental engineering model, even if it is a state-of-the-art model. The attainable degree of accuracy of modelling varies according to the type of simulation. For sewer flow calculations, the accuracy is usually high; for estimation of stormwater runoff, the accuracy is reasonable; and for estimation of pollutant loadings from stormwater runoff (and combined sewage), the accuracy is comparatively lower. The natural processes that influence the generation and transport of pollutants are not yet fully identified or understood. No mathematical formulas can incorporate all the influencing factors of the processes. Even if the formulas could do so, it is not practical to collect all the required input data and to find a suitable computer system to do the computation. Consequently, many simplifying assumptions are still needed in pollutant loading modelling. The above discussion is not to discredit the usefulness of modelling, but is an acknowledgement of the need to interpret modelling results intelligently in the light of the limitations of this technology, which is the best available today.

It is felt more appropriate to interpret pollutant modelling results in the context of long-term statistics than on a single event basis, and in the context of noting the changes in results as simulated conditions change than taking the numerical results too dogmatically.

5.2 Selection of Model

Many simulation models exist in the private market and public domain. Only those models that had received some formal evaluation by a reputable agency were considered for this study, as it was beyond the study scope to evaluate models and it was unwise to use a model that had not been assessed critically.

Eighteen major models had been assessed by the U.S. Environmental Protection Agency (Brandstetter, 1976). Four of the models, namely,

Battelle Urban Wastewater Management Model, Dorsch Quantity/Quality Simulation Model, EPA Stormwater Management Model, and Hydrocomp Simulation Program, met the basic requirements of this CSO study with respect to catchment hydrology, sewer hydraulics and wastewater quality simulation. The first two named models are proprietary. The Hydrocomp model requires extensive resources support that was neither justifiable nor available to this study. The U.S. EPA Stormwater Management Model (latest SWMM Version III.3) was used. In any case, SWMM is the most widely tested and used model both in research and engineering practice for this type of studies.

5.3 Description of the Study Model

The SWMM model consists of 6 blocks each being capable of performing specific functions. Two of the blocks, the Executive and the Runoff, are most often used in most projects using SWMM. In this CSO study, three blocks, the Executive, the Runoff and the Transport were used.

The Executive block is a program manager. The Runoff block is responsible for all computations relating to runoff quantities and qualities and their routing through catchments. The Transport block merges input DWF and infiltration quantities and qualities with runoff results to simulate combined sewage and then routes the combined sewage through the sewer system. Combined sewer overflow is simulated by this block. The algorithms used by the model are discussed at length in the user manual of the model (Huber, 1981).

The SWMM model was set up, or "personalised", to represent the study system by inputting catchment and sewer data and by providing coefficient values of certain mathematical functions to the model to enable the model to estimate flow quantities and pollutant concentrations. The 8 pollutants listed in the study scope were studied.

The following paragraphs outline briefly the model that was set up. Further details are in Appendix C1.

Sewers and Humber WPCP

Trunk sewers downstream of the regulators and the regulators themselves were included in the model. The WPCP was represented by the limiting capacity of 11.8 m^3/s. An assumption made was that any primary effluent in excess of the secondary treatment capacity of 9.6 m^3/s would bypass the secondary treatment process to the effluent outfall after chlorination. Figure 5.1 is a logic layout of the sewer system simulated.

Flow Quantity Calculation

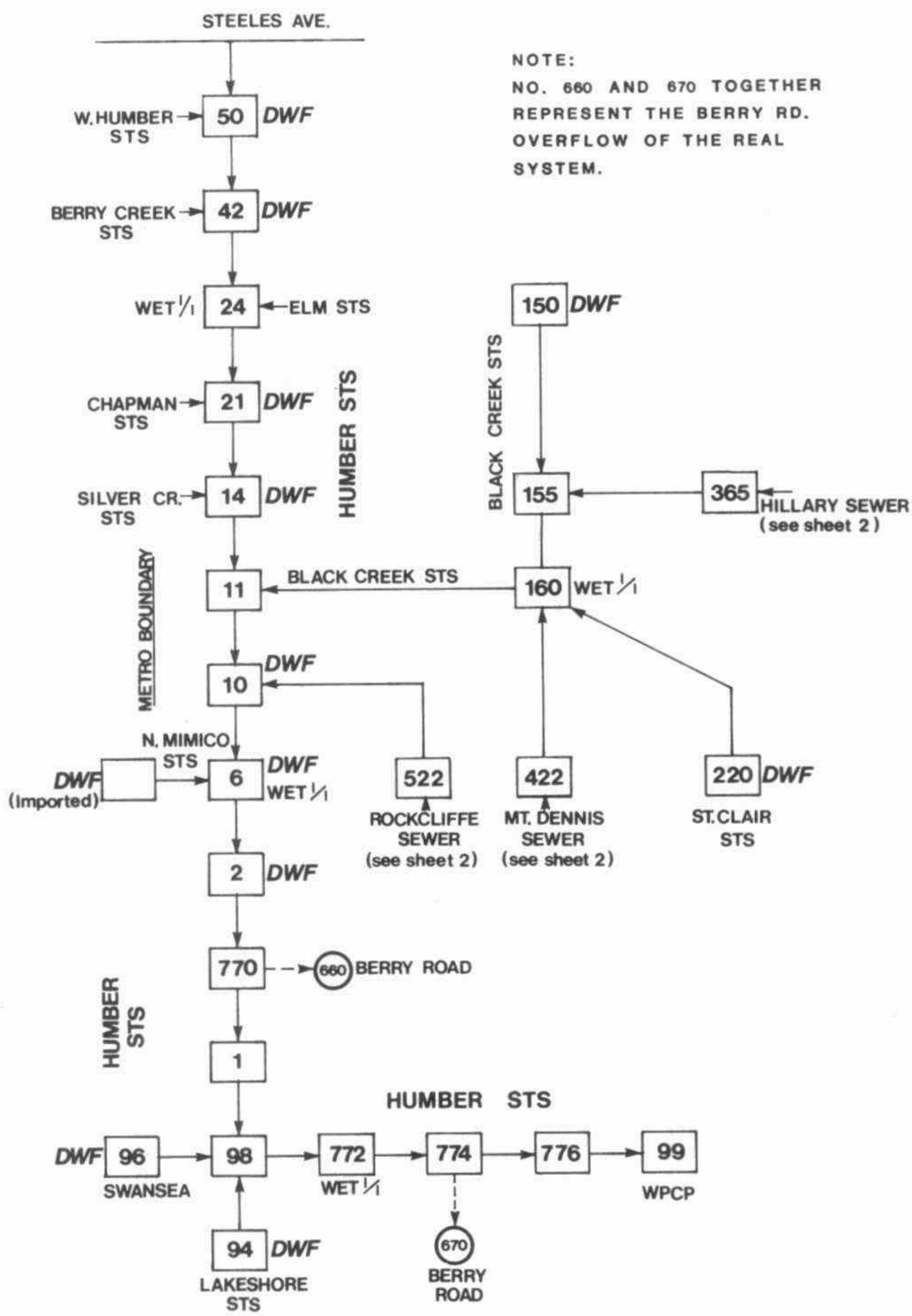
The combined sewer area was represented in the model by 26 subcatchments. The input data included sub-catchment size, percent of impervious surface, ground slope, catchment width, soil moisture coefficients, depression storage and the "family tree" of the sub-catchments for flow routing. Runoff quantity calculation involved the application of the kinematic wave theory, Horton soil moisture equation, and the linear reservoir theory. The calculation process was able to account for the effects of catchment topography; loss of precipitation through infiltration and depression storage; recovery of infiltration capacity and depression storage in a dry period; and change in precipitation intensity.

DWF quantities and qualities of both the combined sewer area and the sanitary sewer area were input, as was wet-weather I/I of the sanitary sewer area. Wet-weather I/I of the combined sewer area was part of the combined sewage.

Runoff Quality Calculation

Input data for runoff quality prediction were derived from the observed combined sewage data. The derivation is in Appendix C2. The following describes briefly the methodology for runoff quality prediction.

Washoff of SS and BOD_5 by stormwater runoff was based on the rating curves shown in Figures 5.2 and 5.3 respectively. The



LEGEND

21 MANHOLE, I.D. NO. 21

WET I/I INLET POINT FOR WET I/I

660 OVERFLOW LOCATION

STS SANITARY TRUNK SEWER

DWF INLET POINT FOR DWF INCLUDING DRY I/I

FIGURE 5.1: STYLIZED LOGIC DIAGRAM OF MODELLED SYSTEM
(sheet 1 of 2)

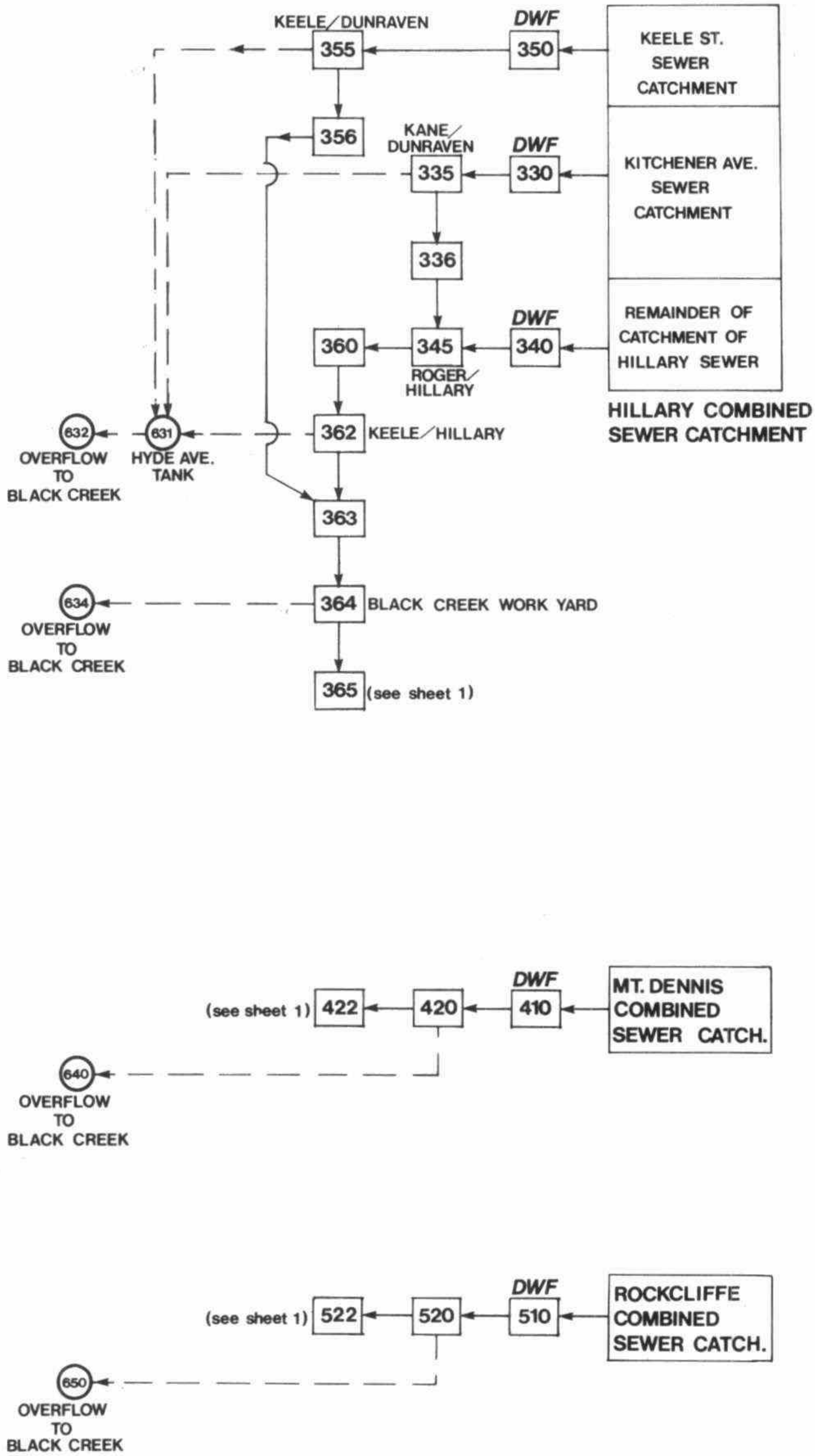


FIGURE 5.1: STYLIZED LOGIC DIAGRAM OF MODELLED SYSTEM

(sheet 2 of 2)

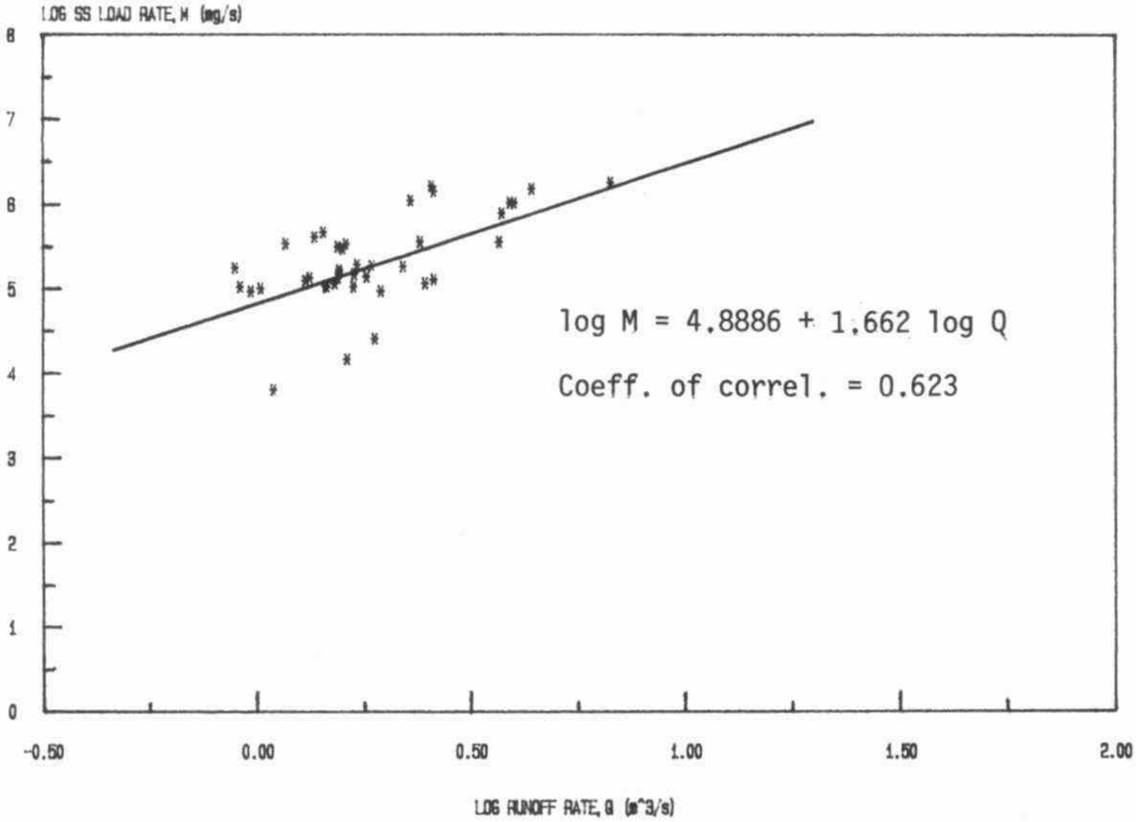


FIGURE 5.2 : SUSPENDED SOLIDS RATING CURVE

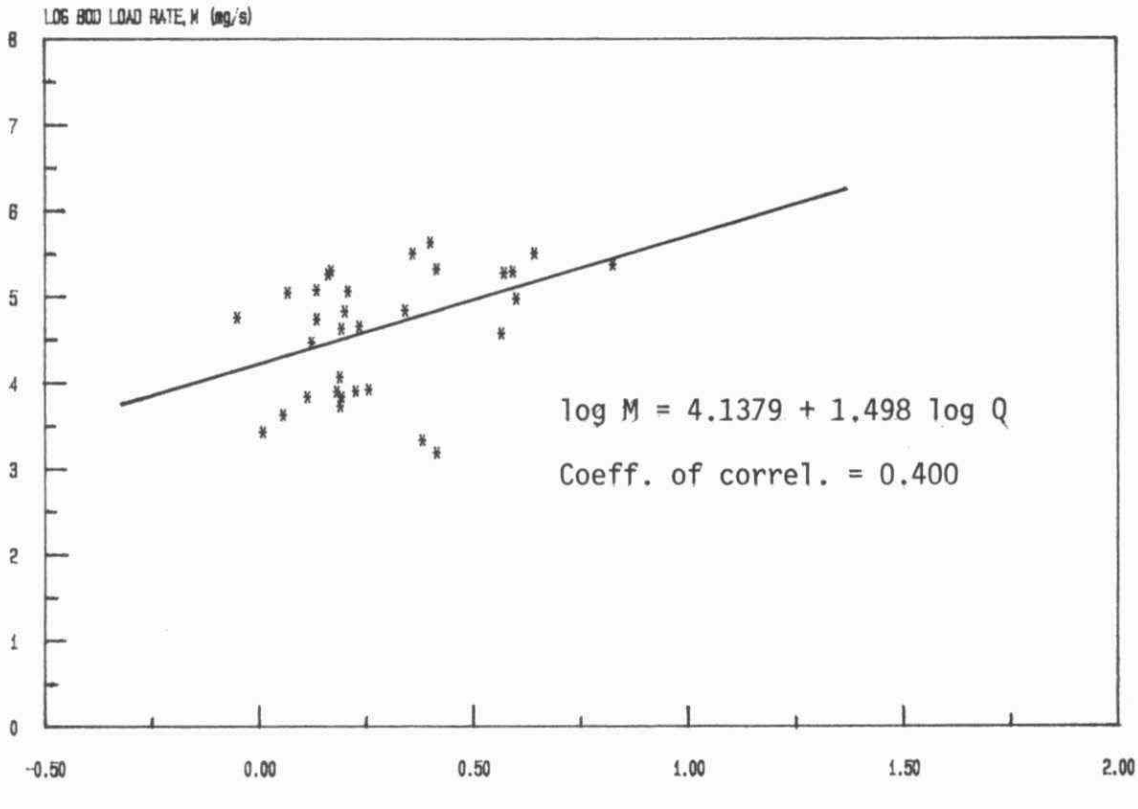


FIGURE 5.3 : BOD_5 RATING CURVE

coefficient of correlation (0.4) for the BOD_5 curve is low. However, since BOD_5 is not a pollutant of great concern to the Humber river, no attempt was made to obtain additional data to produce a better curve.

Washoff of the four heavy metals and total phosphorus was related to SS washoff in the following ratios:

	Total Phosphorus	Cadmium	Copper	Lead	Zinc
Ratio of Pollutant to SS (mg/gm)	9.17	0.038	0.645	1.125	1.886
Coeff. of Variation	51%	110%	65%	60%	48%

The derived runoff concentrations of soluble phosphorus indicated that they were not correlated with the runoff rates. As a result, the derived averaged concentration of 0.54 mg/l was used.

CSO fecal coliform loading was estimated by using the observed combined sewage geometric mean concentration of 1.65×10^6 counts/100 ml. It was believed that there was yet no credible method and data for forecasting the rate of fecal coliform production on a catchment or the rate of washoff from the catchment. Fecal coliform concentrations are influenced by ambient temperature, light intensity, nutrient conditions and possibly some other biological factors but it is yet not possible to identify and quantify all the major influences. Recent Ontario studies (Gore & Storrie, 1981; Gore & Storrie, 1984) indicated the difficulty even to completely identify the sources of observed fecal coliform loadings. Therefore, although superficially sophisticated methods exist for forecasting fecal coliform washoff rates, the use of the observed mean concentration was preferred.

The amount of pollutant washoff was limited by the amount of the pollutant accumulated on the catchment surface. SS accumulation rates were based on Figure 5.4. Limiting accumulation of BOD_5, total phosphorus and the four heavy metals was related to the SS accumulation curve by the ratios mentioned earlier. As soluble

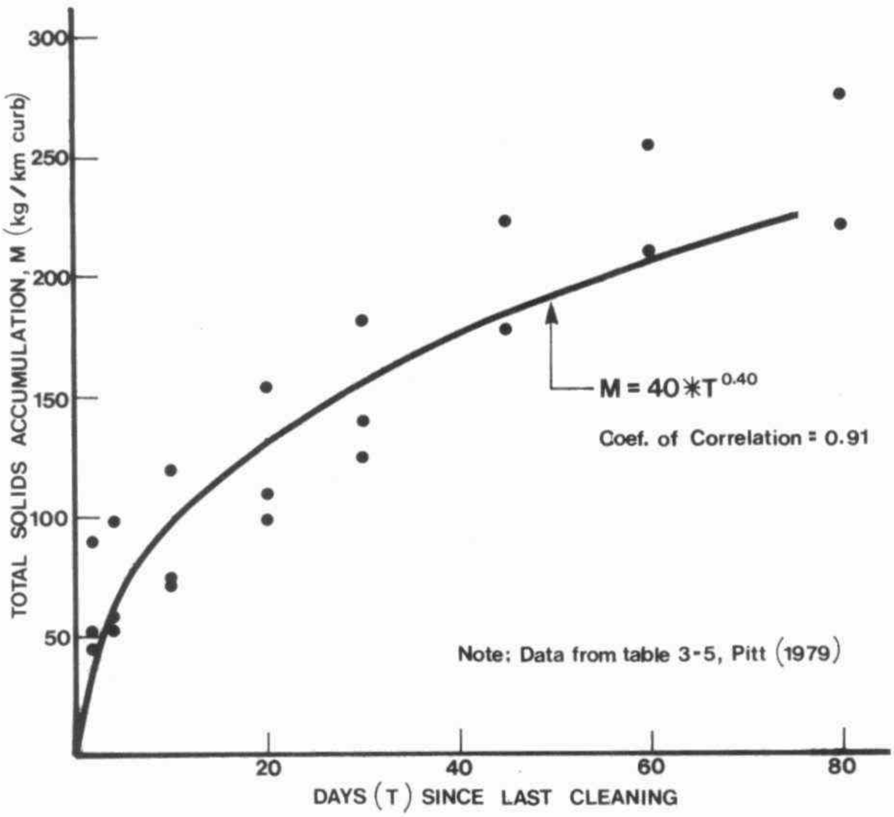


FIGURE 5.4: TOTAL SOLIDS ACCUMULATION

phosphorus concentration was not correlated with stormwater runoff, it was inferred that it was also not related to accumulation. Fecal coliforms were assumed to be plentiful on the surfaces of catchments and would not be depleted by stormwater washoff.

5.4 Model Calibration: The Prevailing Practice

In principle, after a model is set up and before it is commissioned, it should be test-run (calibrated) to check if predicted results for selected storm events agree reasonably with observed data of the same events and values of model parameters should be adjusted as necessary. In practice, however, a model is often applied without calibration (NCASI, 1982).

There is yet no standardized practice in model calibration. A few examples will illustrate the variety of approaches used. In a combined sewer study in New Haven using SWMM (Cermola, 1979), one storm event each was used for calibration and verification. In a Hamilton combined sewer study using the Runoff block of SWMM (James, 1980), 4,000 ha out of a 6,800 ha study area were calibrated for flow quantity. In an Atlanta stormwater management study (Holbrook, 1976), the observed flow quality data used consisted of grab samples and composite samples taken from the beginning of overflow to a point "well beyond the first flush".

The goodness of calibration is often judged by comparing the observed and predicted hydrographs visually. Some recent studies compared the sum of observed and predicted values (Gore & Storrie, 1980; Marshall, Macklin, Monaghan, 1982). The use of statistical measures to judge the goodness is not common but it has been advocated as a more scientific approach (NCASI, 1982).

5.5 Flow Quantity Calibration

In this CSO study, flow quantity calibration was carried out with the Hillary catchment which makes up 72% of the total combined sewer area. Eight of the observed storms in 1983 (Table 4.6) were used for calibration. They were chosen to represent three ranges of

precipitation: below 10 mm per event; between 10 and 20 mm; and over 20 mm. For precipitation, data observed at the Castlefield Works Yard were used.

The "goodness" of calibration was measured by the linear regression equation of predicted event runoff volumes vs observed event volumes and the following three criteria for the measurement were proposed:

(1) The slope of the regression line to be within 0.9 and 1.1.

(2) The intercept of the regression line with the horizontal or vertical axis to be within - 2,690 m^3 and + 2,690 m^3.

(3) The coefficient of correlation of the regression line to be within 0.9 and 1.0.

The meaning of the criteria is illustrated in Figure 5.5. The ideal case, Figure 5.5(A), represents the ideal but unattainable situation in which every predicted event volume agrees exactly with the observed volume of the corresponding event. The slope of the line is 1.0, the intercept is 0 and the coefficient of correlation is 1.0. The regression line is a measure to see how far the actual case deviates from the ideal case.

If the actual regression line tilts upwards from the ideal line, as in Figure 5.5(B), the model will err on the high side and the absolute magnitude of the error increases with larger events. The error will be on the low side if the line tilts downwards.

If the actual regression line is parallel to the ideal line, but intercepts an axis at, say, 2,000 m^3 as illustrated in Figure 5.5(C), the predicted volume is 2,000 m^3 smaller than the true volume. The intercept represents a constant error no matter what the size of the event is.

Hypothetically, the ideal line could be obtained with very scattered points as illustrated in Figure 5.5(D), but the coefficient of correlation will be much smaller than 1. If the coefficient of

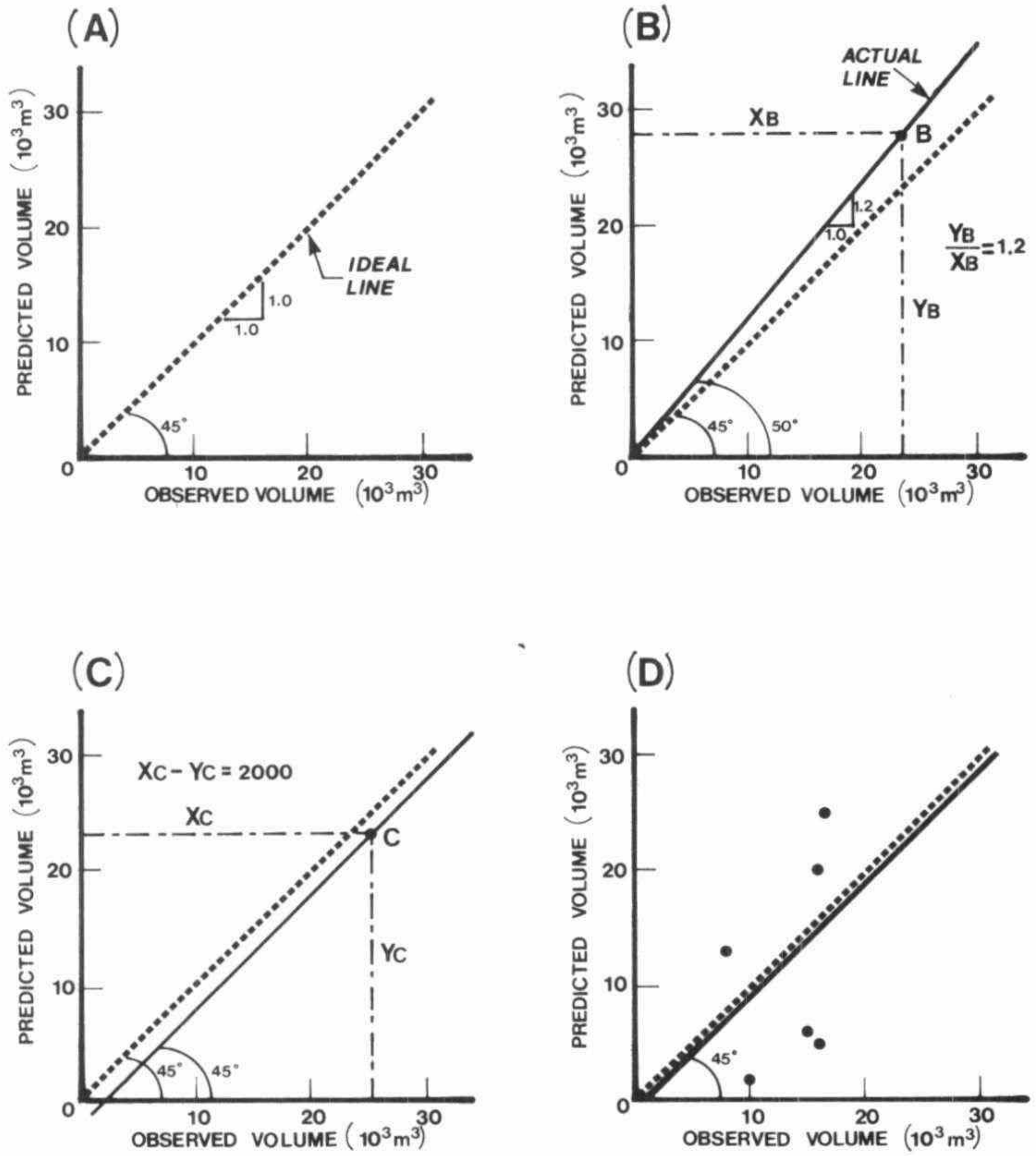


FIGURE 5.5: MEANING OF CALIBRATION CRITERIA

correlation is small, the chance of having a point falling exactly on the line is not high.

The proposed criteria can now be interpreted in the light of the foregoing discussions. Criterion (1) restricted the variable error to 10% of the true event volume. Criterion (2) restricted the constant error to 2,690 m^3 which was 10% of the average observed DWF of the Hillary catchment. Criterion (3) was to ensure that there would be a reasonable chance for the relation between observed and predicted volumes to be truly represented by the regression line.

The criteria were arbitrary, but it will be realized that they were fairly stringent for stormwater modelling.

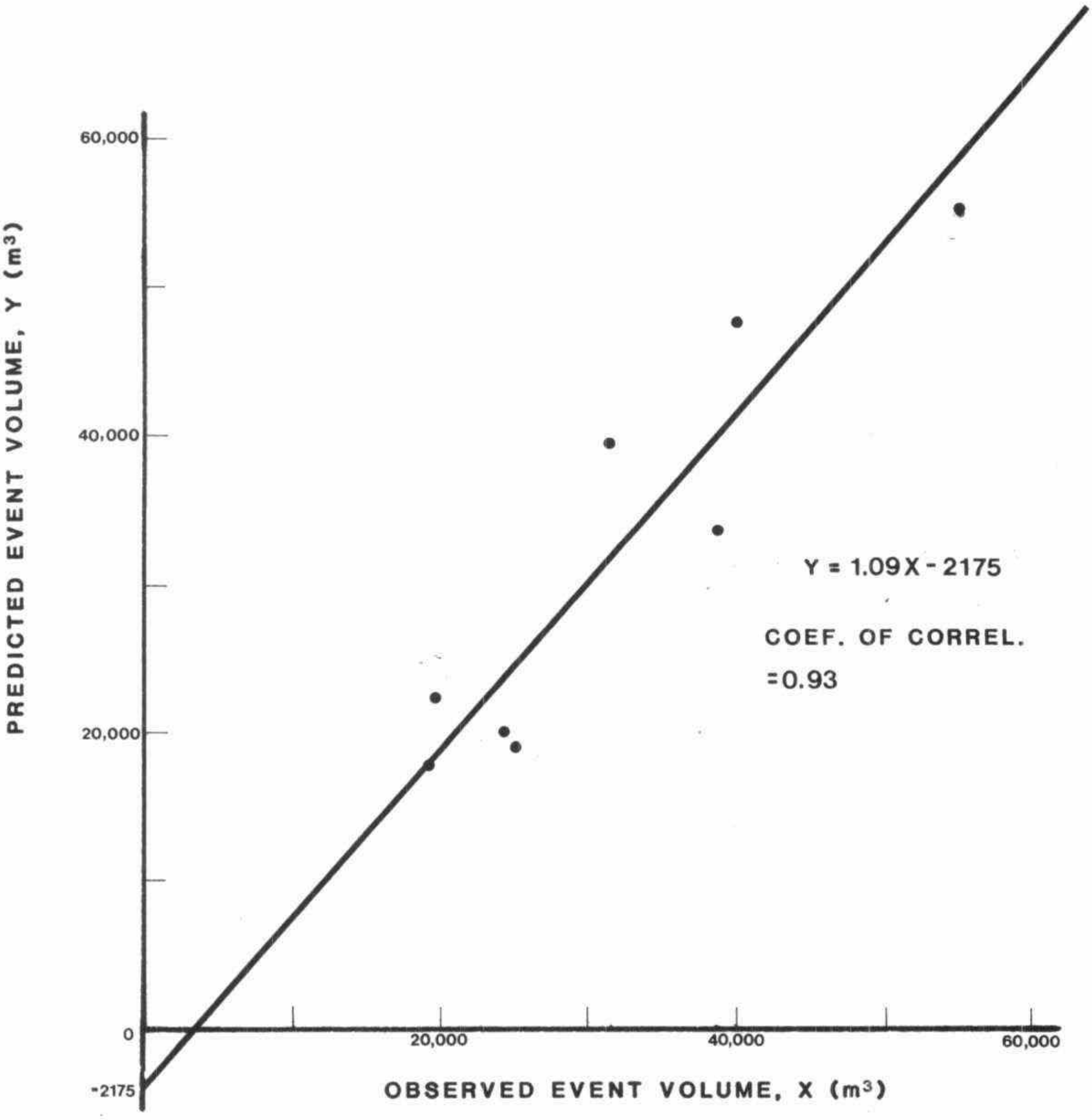
The actual calibration results are shown in Figure 5.6. As all the three criteria were satisfied, the model was considered calibrated. The hydrographs of the calibration events are in Appendix C3.

The model was verified with 3 other observed events. The results are shown in Table 5.1. It can be shown that there is no difference between the predicted and observed results at 90% confidence level according to a student t statistical test (Mendenhall, 1978). So the model was verified.

5.6 Flow Quality Calibration

Flow quality calibration of the study model differed from flow quantity calibration in that each coefficient used in the mathematical expressions for computing accumulation and washoff of each pollutant was derived from observed data and then input to the model. Therefore, the input data in effect calibrated the model. This approach was also used elsewhere (Marshall, Macklin, Monaghan, 1982).

To illustrate that the model was indeed calibrated for predicting flow qualities, the predicted and derived SS concentrations (Table 5.2) were put to the student t test (Mendenhall, 1978). It can be shown that there was no difference between the two sets of data at 90% confidence level.



EVENT	830430	830501	830502	830519	830522	830606	830921	831013	
X	39824	25076	38820	54632	19469	19280	24298	31666	SUM = 253065
Y	48335	19308	33700	55666	22300	18172	21570	39436	SUM = 258487

FIGURE 5.6: CALIBRATION OF FLOW QUANTITIES

TABLE 5.1

QUANTITY VERIFICATION OF MODEL

Event Date	Predicted Event Volume (m^3)	Observed Event Volume (m^3)
830529	26,409	21,653
830704	9,265	8,363
830822	48,082	43,996

TABLE 5.2

QUALITY VERIFICATION OF MODEL (SUSPENDED SOLIDS)

Event Date	Predicted Runoff Flow-wt Conc. (mg/l)	Observed Runoff Flow-wt Conc. (mg/l)
830519	137	151
830529	146	255
830606	88	133
830704	93	107
830822	185	269
830916	214	122
831013	215	157

5.7 Selection of Simulation Period

In practice, when a model as complex as SWMM is used for planning studies, simulation is run often for a period of not more than one year (Marshall, Macklin, Monaghan, 1982; Dorsch Consult Ltd, 1979). In the present CSO study, simulation was done for a selected season (April to October) of a selected year (1979). The rationale for selecting the simulation period is explained below.

Typically, CSO is most pronounced in the summer, because summer storms have high intensities. On the average, a scheme designed for CSO control in the summer (April to October) will be able to control CSO in the remainder (November to March) of the year. An earlier CSO modelling study (Dorsch Consult Ltd., 1979) also made use of this seasonal characteristic to simulate for the April-October season only.

Second, urban hydrology of frozen ground and snow-melt is still in the developmental state and few data are available (Waller, 1974(?)). A Hamilton CSO study using SWMM (Robinson, 1981) also cited this technical limitation as a reason for simulating for the April-October season only.

Finally, historical precipitation data needed for model input were collected for the April-October season only by the Atmospheric Environmental Services (AES) of Environment Canada.

The selection of year 1979 was based on precipitation statistics.

Three AES precipitation stations (Toronto Downsview, Toronto Old Weston Road and Toronto Etobicoke) closest to the combined sewer area were considered to determine how the precipitation recorded at the stations should be weighted to allow for uneven distribution of precipitation over the combined sewer area. Based on the Thiessen method (Linsley, 1975), it was concluded that precipitation in the combined sewer area should be represented solely by the Old Weston Road station data.

The Old Weston Road station had complete data for the April-October season for 16 years from 1966 to 1981, recorded at hourly intervals. A computer program PRCPSTAT was developed in this study to perform the statistical analysis. The precipitation statistics of the April-October season of 1979 (Table 5.3) compared most closely to the average statistics of the 16-year period, so year 1979 was selected.

Huge efforts would be required to do simulation for each of the 64 storm events in the season, because the Transport block of SWMM can handle one event only in each run of the computer program. As an alternative, as shown in Table 5.4, each event with precipitation more than 10 mm and one event each from the next three lower ranges of precipitation were simulated. To produce statistics for the season, the simulation results for the groups of storms with 8-10 mm, 6-8 mm, and 4-6 mm of precipitation were multiplied by the group factors 5, 3 and 5 respectively. Therefore, 27 events were in effect simulated.

The remaining 40 storm events in the ranges of not greater than 4 mm of precipitation were ignored. They yielded a total precipitation of 54 mm which was 13 % of the season's total. The total runoff produced by these minor storm events was expected to be less than 13% because minor events had proportionally higher precipitation loss to evaporation and so forth than larger events. Available observed data (Table 5.5) indicated that overflow in minor storms (not more than 4 mm precipitation) was mostly marginal. It was expected that CSO in minor storm events would be readily eliminated with the minimum CSO control. Ignoring the minor storm events actually had the advantage of eliminating distortion of CSO frequency statistics.

With completion of the pre-requisites, the study model was ready for application.

TABLE 5.3

PRECIPITATION STATISTICS OF AES OLD WESTON ROAD STATION

	Seasonal Average of 1966-1981 Period				1979 Season		
	Per Event		Per Season		Per Event		
	Mean	Std Dev	Mean	Std Dev	Mean	Std Dev	Seasonal Total
Number of Events			64				68
Precip. Duration (hr)	6	7.5	410	68.7	6	7.2	463
Precip. Depth (mm)	6.9	9.74	447.7	81.19	6.3	8.23	434.5
Avg. Intensity (mm/hr)	1.1	1.71			1	1.26	
Ante. Dry Duration (hr)	72	84.3	4620	106.1	67	72.4	

Note : Season is from April 1 to October 31.

TABLE 5.4

SELECTION OF STORM EVENTS FOR SWMM APPLICATION

Event Precipitation	Number of Occurrences Of Events	Number of Events Selected
(1) Greater than 10 mm	15	15*
(2) Above 8, not exceeding 10 mm	5	1**
(3) Above 6, not exceeding 8 mm	3	1**
(4) Above 4, not exceeding 6 mm	5	1**
(5) Above 2, not exceeding 4 mm	12	0
(6) Not exceeding 2 mm	28	0
Total	68	18

Notes:

* Two events in this group occurred on the same day. The two events were treated as one in SWMM simulation.

** Multiply simulation results of event classes (2), (3) and (4) by factors 5, 3 and 5 respectively to calculate seasonal statistics.

TABLE 5.5

OBSERVED OVERFLOW DATA FOR MINOR STORMS (1)

Date	Precip(mm)	Site 3 Peak Overflow Rate(m3/s)	Mt. Dennis Peak Overflow Rate(m3/s)	Rockcliffe Peak Overflow Rate(m3/s)
83/04/11	3.2			
83/04/21	.3			
83/04/27	.7			
83/05/04	3.7	.2	.08	
83/05/08	.3	.021	.033	
83/06/03	3.2	.119	.012	.111
83/06/05	1			.038
83/06/30	1.2			.023
83/07/21	2			.085
83/07/28	2.7			.525
83/07/29	.5	.053		
83/07/31	3	.172		1.286
83/08/01	1			
83/08/03	3			
83/08/05	1.7		.001	
83/08/27	2.8	.545	.104	.658
83/08/30	3.7	.2	.033	.122
83/09/06	.5			.019
83/09/09	.3			
83/09/20	1.5			.111
83/09/22	.5			
83/09/23	1.7			
83/09/25	1.9			
83/10/03	2.2			
83/10/04	1.9	.095		
83/10/05	3.7	.053		

Note:

(1) Event with less than 4 mm of precipitation.

6.0 THE BASE CASE

6.1 Seasonal CSO Statistics

The base case analyzed the response of the existing (year 1983) catchments and sewer system in wet weather. The weather conditions were represented by the precipitation data of the selected season, April-October, 1979. The results of the base case provided a direction for formulating CSO control schemes. For example, the results could be used for determining the priority of control of regulators or catchments, and the desirable magnitude of threshold capacities. The results also provided the basis for comparing the effectiveness of the control schemes in CSO reduction. A Summary of results is presented below; detailed results are in Appendix D1.

Estimated CSO frequencies of the regulators are shown in Figure 6.1. Rockcliffe overflowed 26 times, Mt. Dennis and Site 3 each 27 times and Hyde Avenue tank 7 times in the study season. Results of the Berry Road regulator will be discussed later. So the first 3 regulators overflowed in almost every storm event having greater than 4 mm precipitation. Hyde Avenue tank overflowed less frequently. This was expected as the tank was built for reducing overflow. However, since Site 3 and the Hyde Avenue tank both belong to the Hillary catchment, the Hillary catchment in effect overflowed in each of the 27 storm events.

The Hyde Avenue tank overflow outlet and the Site 3, Mt. Dennis and Rockcliffe regulators all discharge to the Black Creek and are located closely together. So, as far as CSO impacts on the Black Creek are concerned, these 3 regulators and the outlet of the tank could be considered as if they were one entity. For convenience, these 4 devices are collectively called the Black Creek group of regulators. One practical significance of the closeness of these devices and the nearly equal CSO frequencies was that, if a control scheme was designed to eliminate overflow in a selected storm event, the scheme should eliminate CSO from the whole group in that event, else the aim of eliminating CSO in that event would be defeated.

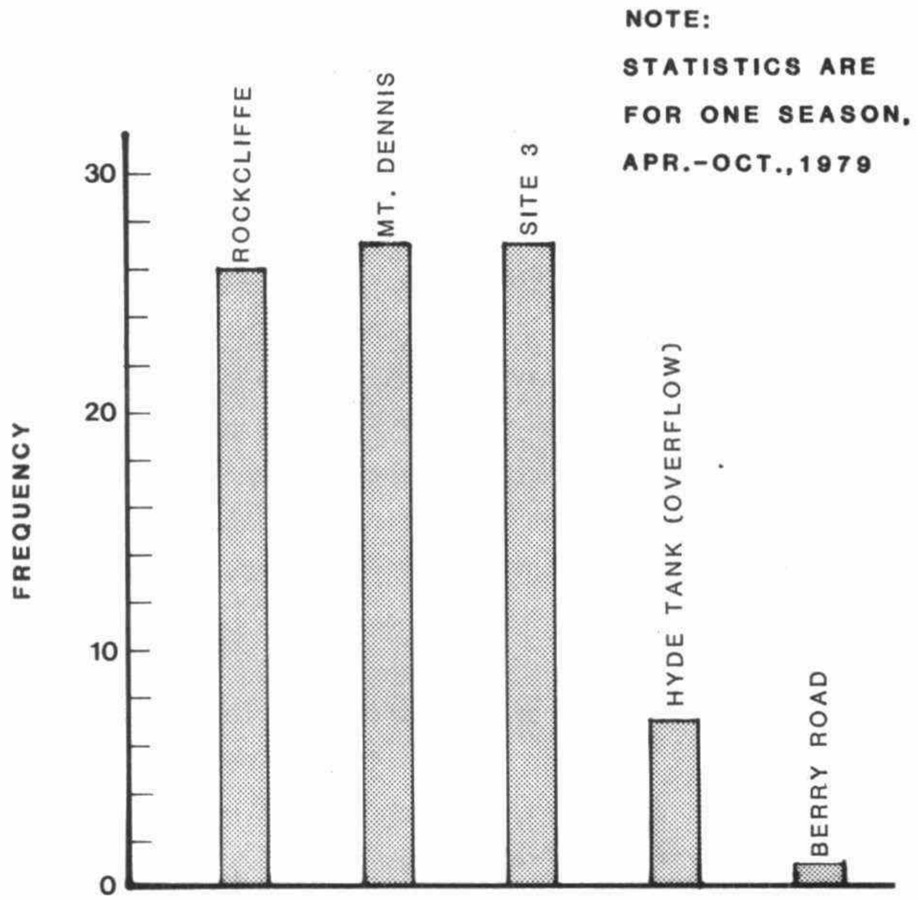


FIGURE 6.1: ESTIMATED OVERFLOW FREQUENCIES
(EXISTING SYSTEM)

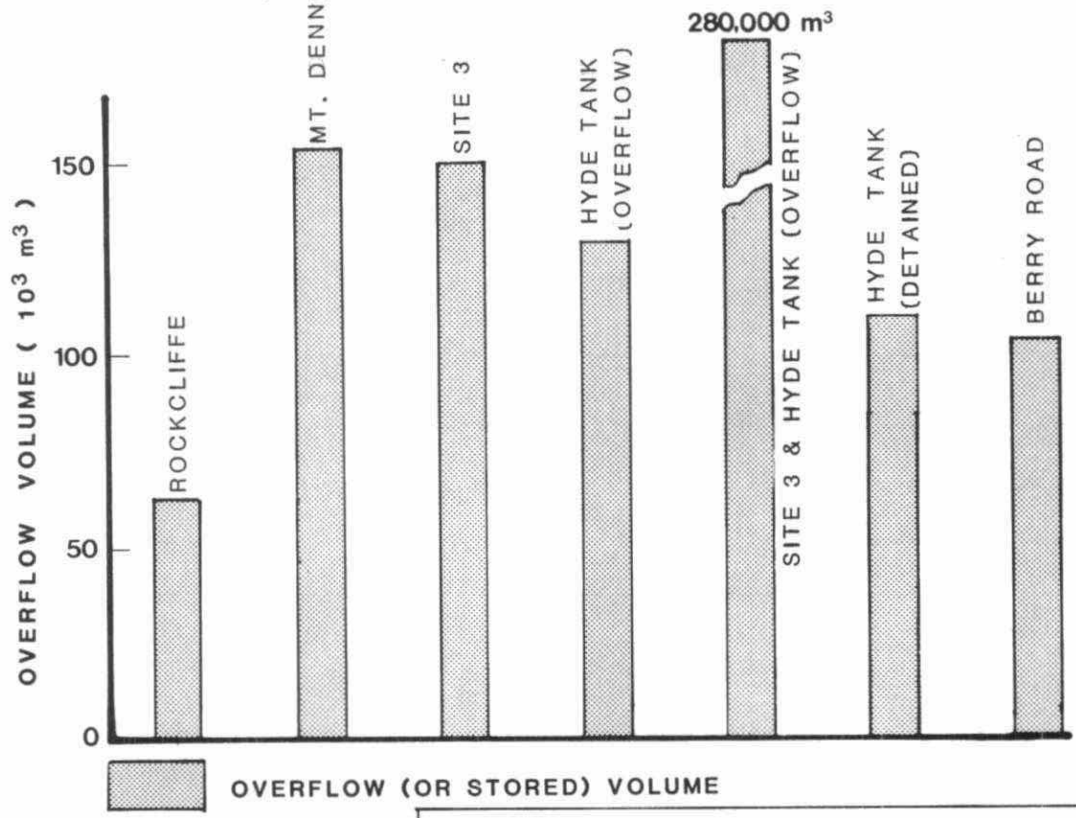
The estimated CSO seasonal volume from the Black Creek group of regulators and the seasonal volume stored in the Hyde Avenue tank are shown in the upper diagram of Figure 6.2. Again, the toal overflow volume from the Hillary catchment should be the sum of overflow at Site 3 and the Hyde Avenue tank. With a volume of 280,000 m^3, the Hillary catchment was the largest CSO contributor.

The Hillary and the Rockcliffe catchments each discharged about 20% of their combined sewage to the Black Creek in the 27 events and Mt. Dennis discharged 40% of its own. The Hyde Avenue tank stored 8% (102,000 m^3) of the combined sewage generated by the Hillary catchment in the 27 events. If this tank were not in existence, the Hillary catchment would have discharged 28% of its combined sewage. The Mt. Dennis discharge of 40% was comparatively high because this catchment has a comparatively higher percentage of surface imperviousness and the regulator's threshold capacity per hectare of catchment is not correspondingly high, as indicated in Table 6.1.

CSO seasonal pollutant loads are summarized in Table 6.2. Seasonal fecal coliform load is omitted because it is a meaningless statistic, but event CSO fecal coliform loads are available (Appendix D1). They ranged from 80,000 billion to 670,000 billion organisms in a single event. The impacts of these pollutants on the Humber river would be studied by another TAWMS project. Time series of CSO flow rates and pollutant concentrations, including fecal coliforms, were provided to that TAWMS project.

The Berry Road regulator is distinct from the Black Creek group of regulators in three ways: it overflowed only in one storm in the season (Figure 6.1); it discharges to the Humber river and not the Black Creek; it is far away (6.1 km) from the nearest regulator (Rockcliffe) of the Black Creek group.

The CSO volume (Table 6.2) from the Berry Road regulator was large, but the storm causing this regulator to overflow was an intense one as will be seen in the next section. The CSO pollutant concentrations at the Berry Road regulator were low. For example,



NOTES:

(1) STATISTICS ARE FOR ONE SEASON, APR.-OCT., 1979

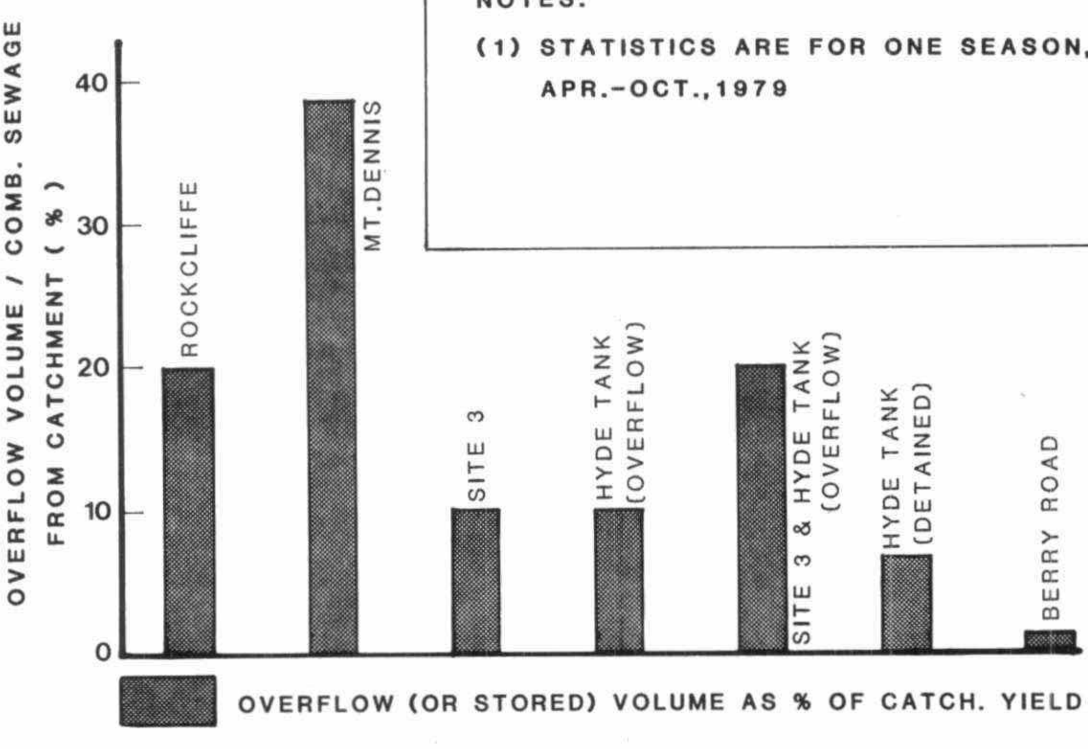


FIGURE 6.2: ESTIMATED OVERFLOW VOLUMES (EXISTING SYSTEM)

TABLE 6.1

COMPARISON OF IMPERVIOUSNESS AND THRESHOLD CAPACITIES

Catchment	Regulator	Catchment Imperviousness (%)	Threshold Capacity of Regulator (m^3/s/ha)
Hillary	Site 3	26.3	.001724
Mt. Dennis	Mt. Dennis	49.0	.001744
Rockcliffe	Rockcliffe	33.5	.002397

TABLE 6.2

ESTIMATED CSO POLLUTANT LOADS FROM REGULATORS
UNDER EXISTING CONDITIONS (1)

	Volume or Load		
Overflow	Black Creek Group Regulators	Berry Road Regulator	Total of All Regulators
Volume	494,000	105,000	599,000
SS	97,000	4,700	101,700
BOD5	23,000	4,900	27,900
Sol-P	320	70	390
Tot-P	1,020	140	1,160
Cadmium	4	0.4	4.4
Copper	61	9	70
Lead	101	1.4	102.4
Zinc	176	5	181

Notes:

(1) Catchments and sewers as in 1983.
Season was April - October, 1979.
Volumes in m^3; loads in kg.

using results in Table 6.2, the average CSO SS concentration was 45 mg/l (4,700 kg/105,000 m^3). Two explanations may be given. First the pollutants on the catchment surfaces were depleted before this intense storm ended. Second, much of the CSO from this regulator was wet-weather I/I from the sanitary sewer area. The I/I was assumed to be free of SS.

There were informal estimates that the Berry Road regulator overflowed more than 20 times in a season and this information was markedly in conflict with the simulation result of one overflow occurrence only. To check the validity of the simulation, two flow monitors were installed in the Humber STS at this regulator in early June 1984 for continuous monitoring until late October 1984. One monitor measured the overflow. The other measured the flow immediately upstream of the regulator. Only one overflow event was observed in the monitored period and it occurred during a thunderstorm (on September 14). The water depth topping the overflow weir crest was less than 10 mm and the overflow duration was about 1 hr. The observation supported the findings of the simulation that the Berry Road regulator would overflow in intense storms only.

6.2 Storm Event on July 11, 1979

The intense storm that caused the Berry Road regulator to overflow occurred on July 11, 1979 (Event 790711). It produced large CSO volumes and pollutant loads which weighed heavily in the CSO seasonal statistics as can be seen in Table 6.3. To eliminate CSO in this event would require containment of a CSO volume of 265,000 m^3, which is large compared with the volume (7,800 m^3) of the existing Hyde Avenue tank or the next largest CSO volume (41,000 m^3) in a single event in the season. To control CSO in Event 790711 would require controlling not only the combined sewer area but also the sanitary sewer area, because the CSO in this event arose from both the Black Creek group of regulators (160,000 m^3) and the Berry Road regulator (105,000 m^3).

TABLE 6.2

ESTIMATED CSO POLLUTANT LOADS FROM REGULATORS
UNDER EXISTING CONDITIONS (1)

	Volume or Load		
Overflow	Black Creek Group Regulators	Berry Road Regulator	Total of All Regulators
Volume	494,000	105,000	599,000
SS	97,000	4,700	101,700
BOD5	23,000	4,900	27,900
Sol-P	320	70	390
Tot-P	1,020	140	1,160
Cadmium	4	0.4	4.4
Copper	61	9	70
Lead	101	1.4	102.4
Zinc	176	5	181

Notes:

(1) Catchments and sewers as in 1983.
Season was April - October, 1979.
Volumes in m^3; loads in kg.

using results in Table 6.2, the average CSO SS concentration was 45 mg/l (4,700 kg/105,000 m^3). Two explanations may be given. First the pollutants on the catchment surfaces were depleted before this intense storm ended. Second, much of the CSO from this regulator was wet-weather I/I from the sanitary sewer area. The I/I was assumed to be free of SS.

There were informal estimates that the Berry Road regulator overflowed more than 20 times in a season and this information was markedly in conflict with the simulation result of one overflow occurrence only. To check the validity of the simulation, two flow monitors were installed in the Humber STS at this regulator in early June 1984 for continuous monitoring until late October 1984. One monitor measured the overflow. The other measured the flow immediately upstream of the regulator. Only one overflow event was observed in the monitored period and it occurred during a thunderstorm (on September 14). The water depth topping the overflow weir crest was less than 10 mm and the overflow duration was about 1 hr. The observation supported the findings of the simulation that the Berry Road regulator would overflow in intense storms only.

6.2 Storm Event on July 11, 1979

The intense storm that caused the Berry Road regulator to overflow occurred on July 11, 1979 (Event 790711). It produced large CSO volumes and pollutant loads which weighed heavily in the CSO seasonal statistics as can be seen in Table 6.3. To eliminate CSO in this event would require containment of a CSO volume of 265,000 m^3, which is large compared with the volume (7,800 m^3) of the existing Hyde Avenue tank or the next largest CSO volume (41,000 m^3) in a single event in the season. To control CSO in Event 790711 would require controlling not only the combined sewer area but also the sanitary sewer area, because the CSO in this event arose from both the Black Creek group of regulators (160,000 m^3) and the Berry Road regulator (105,000 m^3).

TABLE 6.3

ESTIMATED CSO POLLUTANT LOADS IN EVENT 790711
UNDER EXISTING CONDITIONS (1)

Overflow	Volume or Load of All Regulators: Event 790711 Alone	Seasonal Total (2)	Event 790711 as % of Seasonal Total (2)
Volume	265,000	599,000	44%
SS	39,000	101,000	38
BOD5	12,000	27,900	43
Sol-P	170	390	43
Tot-P	460	1,160	40
Cadmium	1.7	4.4	40
Copper	29	70	41
Lead	38	102.4	37
Zinc	68	181	38

Notes:

(1) Catchments and sewers as in 1983.
Season was April - October, 1979.
Volumes in m^3; loads in kg.

(2) Seasonal total including
Event 790711

In order to arrive at a strategy for handling the CSO in this event, the characteristics of this storm were analyzed. The cause of the large CSO volume produced was that 28.1 mm of precipitation concentrated in one single hour and 35.9 mm in two consecutive hours. The recurrence intervals of these two intensities within the given durations were both 3.3 years as shown in Table 6.4. In contrast, the corresponding recurrence intervals of the storm (June 10, 1979) producing the next largest CSO volume (41,000 m^3) were 0.9 and 0.7 year respectively and the recurrence interval of the average intensity was 1.8 years.

In conclusion, the CSO study considered that Event 790711 was an infrequent storm and it was decided that the development of CSO control schemes should exclude consideration of Event 790711. Consequently, all CSO simulation results presented from this point on exclude Event 790711. The base case results presented earlier were adjusted to exclude Event 790711 and the adjusted results are given in Table 6.5. Note that when this event was excluded, the Berry Road regulator did not overflow in the season and that the CSO statistics all pertained to the Black Creek group regulators.

6.3 Treatment Loads on Humber WPCP

Averaged over the season, the volume of sewage treated in the Humber WPCP in the base case was 0.526 m^3/person/d (including combined sewage) for the combined sewer area and 0.735 m^3/person/d for the sanitary sewer area as shown in Table 6.6. It may be inferred that the treatment cost per person was lower for the combined sewer area than the sanitary sewer area, despite the flow of some combined sewage to the WPCP.

In wet weather, the combined sewer area required a higher treatment capacity per person than the sanitary sewer area. The required capacities were .0000298 m^3/person/s and .0000164 m^3/person/s respectively (1.8:1.0) as shown in Table 6.7.

TABLE 6.4

COMPARISON OF PRECIPITATION DATA OF STORM ON JULY 11, 1979

	Event on July 11, 1979		Event on June 10, 1979	
Attribute	Actual Value	Recurrence Interval (year)	Attribute Value	Recurrence Interval (year)
Event precipitation	36.4 mm	0.6	17.4 mm	<0.3
Event duration	7 hr	N.C.	3 hr	N.C.
Ante. dry period	17 hr	N.C.	61 hr	N.C.
Avg. event intensity	5.2 mm/hr	1.6	5.8 mm/hr	1.8
1 hr max. precip.	28.1 mm	3.3	16.2 mm	0.9
Max. precip. in 2 consecutive hours	35.9 mm	3.3	16.4 mm	0.7
Max. precip. in 4 consecutive hours	36.1 mm	2.9		
Max. percip. in 6 consecutive hours	36.2 mm	2.0		

Note: N.C. = Not calculated.

TABLE 6.5

ADJUSTED CSO POLLUTANT LOADS FROM REGULATORS
UNDER EXISTING CONDITIONS (1)

Overflow	Adjusted Seasonal Total Volume or Load of Black Creek Group Regulators
Volume	334,000
SS	63,000
BOD5	16,000
Sol-P	230
Tot-P	690
Cadmium	2.6
Copper	41
Lead	65
Zinc	112

Notes:

(1) Catchments and sewers as in 1983.
Season was April - October, 1979.
Volumes and loads adjusted to exclude Event 790711.
Volumes in m^3; loads in kg.

TABLE 6.6

SEWAGE VOLUMES TREATED IN HUMBER WPCP - EXISTING CONDITIONS (1)

	Estimated Volume to Humber WPCP (m3)	
	Comb. Sewer Area	San. Sewer Area
Storms Greater Than 4 mm	1,550,000	7,340,000
Storms Not Greater Than 4 mm (2)	320,000	0
Volume Returned by Hyde Avenue Tank	100,000	0
Flow in Dry Days	7,180,000	63,220,000
Total in Apr-Oct, 1979 (3)	9,150,000	70,560,000
Volume/Person/Day (4)	0.526	0.735

Notes:
(1) Catchments and sewers as in 1983.
Season was April - October, 1979.
(2) Runoff Volume. Assumed to be 0.4 of precipitation.
(3) Excluding event 790711.
(4) Population:
Combined Sewer Area 81,274
Sanitary Sewer Area 448,931

TABLE 6.7

MAXIMUM FLOW RATES TO HUMBER WPCP
UNDER EXISTING CONDITIONS (1)

	Max. Flow Rate from Sewer Area (m^3/s)	Max. Flow Rate Per Person in Sewer Area (m^3/s/person) (2)
Combined Sewer Area	2.43	.0000298
Sanitary Sewer Area	7.37	.0000164
Total of Both Areas	9.80	.0000185
WPCP Max. Capacity	11.8	.0000223

Notes:

(1) Catchments and sewers as in 1983. Maximum flow rate estimated for precipitation data in April-October, 1979. Event 790711 excluded.

(2) Population:
Combined sewer area 81,274.
Sanitary sewer area 448,931.

If the peak primary treatment capacity of the Humber WPCP should, in fact, be taken as 8.9 m^3/s as mentioned in Section 3.4, then the peak primary treatment capacity was exceeded in 7 storm events in the base case. However, the maximum excess in capacity was only 0.9 m^3/s (9.8-8.9) which was 10% of the revised capacity, and the duration of each excess was only a fraction of the storm duration. Therefore, it could be considered that the WPCP had sufficient capacity to treat all the wet weather flow it received.

7.0 CSO CONTROL SCHEMES

7.1 Introduction

While many methods for CSO control are mentioned in the literature, not many methods have been sufficiently evaluated. A co-author of a recent U.S. Environmental Protection Agency study of case histories of urban stormwater management and technology observed that "measures to control pollution by intermittent stormwater discharges and combined sewer overflow are still in early stages of implementation. Few of these have adequate performance data available." (Finnemore, 1982).

By far the control methods most often used have been structural measures. They include: on-site treatment; off-line and on-line storage; and increase in support in treatment by dry-weather treatment facilities. The above-mentioned U.S. project which studied full scale applications of these measures in Seattle (Washington), Saginaw (Michigan) and Mt. Clemens (Michigan) concluded that the measures, when applied in suitable combinations, were very promising. (Finnemore, 1982).

Most commonly cited examples of on-site treatment include microstraining, air floatation and the swirl concentrator for removal of solids; aerated lagoons for removal of conventional pollutants; chemical coagulation for removal of particulate substances; and disinfection for reduction of bacteria. Although many of these processes are well recognized in the treatment of DWF and show potential for application to treating intermittent flows in pilot-scale studies (Kronis, 1975(?) and 1982; Nebolsine, 1972; Prah, 1979), they are not often used in full-scale application. Possible reasons for their limited use may include high capital cost per kilogram of pollutant removed; uncertainty in results; uncertainty in operational reliability; and the trend of centralizing treatment facilities. Nevertheless, on-site CSO disinfection and screening would be studied in a separate TAWMS project.

"Best management practices" such as street sweeping, catchbasin cleaning and sewer cleaning do not reduce CSO frequency or volume. They are not expected to produce appreciable reduction in pollution loads to the Humber river when applied to the combined sewer area, because the area is only 7% of the Humber river watershed in Metro Toronto. For these reasons, they were not considered.

The following CSO control schemes were developed and considered in this study:

Scheme 1: <u>Detention of overflow at regulators</u>.

Scheme 2: <u>Resetting of regulators</u>,
aided/unaided by detention at regulators.

Scheme 3A: <u>Stormwater runoff control (20% catchment reduction</u>)
aided/unaided by detention at regulators.

Scheme 3B: <u>Stormwater runoff control (28% catchment reduction</u>)
aided/unaided by detention at regulators.

The following 4 alternative methods for reduction of catchment areas were considered in Scheme 3A:

Scheme 3A(i) <u>Detention tanks in local sewers</u>

Scheme 3A(ii) <u>Roof leader disconnection</u>

Scheme 3A(iii) <u>Combined sewer separation</u>

Scheme 3A(iv) <u>Catchbasin inlet restriction</u>

Schemes 1, 2, 3A(i), (ii), (iii) and 3B were found feasible; Scheme 3A(iv) was not. Each of the feasible schemes is self-contained and is an alternative, but not a supplement or a complement, to the others. Each feasible scheme was analyzed to determine the capacities required for various degrees of CSO control, up to complete CSO elimination in the season of April-October, 1979. Note that all schemes considered

Event 790711 as an exception. The maximum recurrence interval of storms in which the CSO control schemes may achieve complete CSO elimination is 1.8 years. All the analyses were carried out with the study model.

In developing the CSO control schemes, engineering judgement was used to see that the schemes were conceptually sound in layout and practical in construction and operation. An order of costs of the schemes was made so that relative merits of the schemes could be evaluated. Unit costs used in the estimates were obtained from the Ministry of the Environment (Braganza, 1985) unless otherwise noted. Costs for engineering services, land and ancillary works such as access road, landscaping of storage sites and instrumentation, were not included in the estimates. A breakdown of the estimates is in Appendix D2. Detailed feasibility study and costing of the schemes would be undertaken by a separate TAWMS project.

The following sub-sections discuss the individual schemes. For convenience of presentation, discussion about the feasibility of integrating measures for basement flooding mitigation into CSO control schemes is reserved until Section 8.0.

7.2 CSO Control Scheme 1: Detention of Overflow at Regulators

This scheme provided new detention storage to intercept CSO from the Black Creek regulators. The storage could be made up of one or more tanks as will be discussed later. The new storage would augment the existing Hyde Avenue tank. In general practice, temporary detention of flow has been widely used in treatment of DWF to equalize the flow and hence to reduce the required peak treatment and sewer capacities. This control method applied to combined sewage should be even more beneficial because the peaks of combined sewage are typically many more times higher than the DWF peak. It was assumed that if the new storage was filled, the excess of the intercepted flow would overflow to the Black Creek. It was also assumed that after a storm event, the detained flow would be returned to the Black Creek STS via an underdrain in the storage facility and it would take 24 hours to empty the full storage. This duration was selected to ensure that the

return flow would not overload the sewers or the Humber WPCP by rapid draining and yet the storage facility would be completely emptied before the next storm came. For a total new and existing storage as large as 50,000 m^3, the return flow rate would be 0.57 m^3/s which was only 12% of the average design secondary treatment capacity of the Humber WPCP and would not stress the WPCP.

The catchments and sewers simulated in this case were the same as the base case, i.e. conditions existing in 1983.

Four alternative arrangements of the new tank(s), as shown in Figure 7.1, were evaluated. The final choice will depend on required storage capacity, engineering feasibility and costs. Arrangement 1 used one new tank situated downstream of the Rockcliffe regulator. The tank intercepted this regulator as well as the Mt. Dennis and Site 3 regulators. The existing and the new tanks were not interconnected. This arrangement would be suitable if the selected degree of control was low enough such that the storm event to be controlled would not cause the existing tank to overflow. Arrangement 2 was similar to arrangement 1 but the new and existing tanks were interconnected to allow overflow from the existing tank to the new tank. This arrangement and arrangements 3 and 4 could be used for CSO control up to complete CSO elimination. Arrangement 3 used 2 new tanks: one situated as in arrangement 1 and the other near the existing tank to intercept the tank's overflow. The two new tanks were not interconnected. Arrangement 4 used 3 new tanks which were not interconnected. Obviously, this arrangement would be costlier than either arrangement 2 or 3. It was included in case site conditions precluded the choice of the other arrangements.

New sewers would be needed to connect each existing overflow regulator with the new tank(s). The sizes of the connectors are given in Appendix D3. They were designed for the maximum overflow rates in the storms to be controlled. It will not be practical nor justifiable to provide connector sewers of very large sizes to cope with more extreme storm events in other seasons. Therefore, an

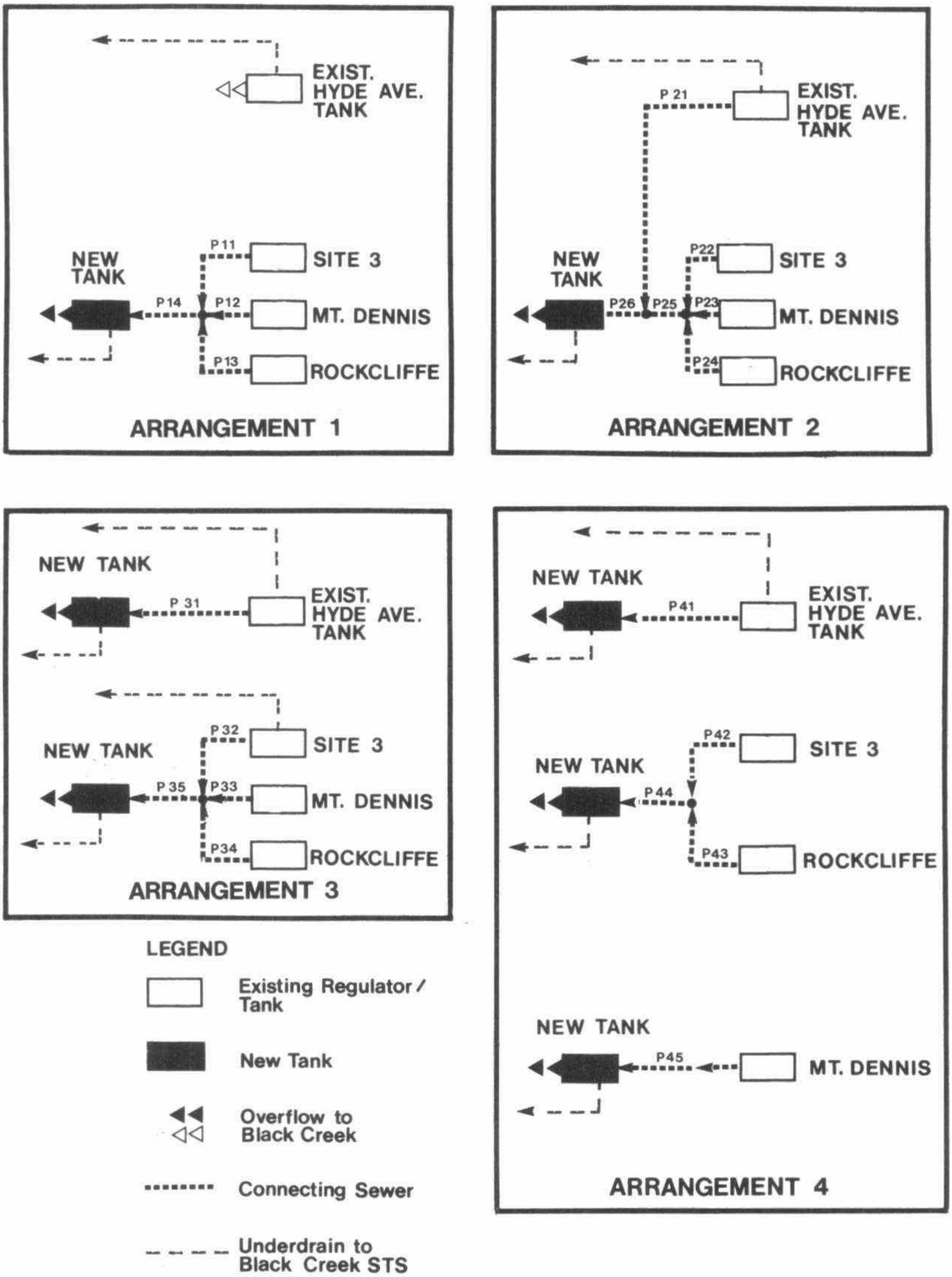


FIGURE 7.1 : ARRANGEMENT OF STORAGE TANKS

emergency overflow opening should still be provided at the upstream end of each connector but the opening should be so designed that overflow will be allowed only when the connector capacity is exceeded.

A series of new storage capacities was examined at increments of 4,000 m^3 until CSO was eliminated completely.

7.2.1 Performance of Scheme 1

The estimated seasonal CSO frequencies against new storage capacities are shown in Figure 7.2. For required new storage up to 17,000 m^3, arrangement 1 would be adequate; the CSO frequency would be reduced from 26 to 7. For required new storage between 17,000 m^3 and 37,000 m^3, arrangements 2 and 3 were comparable in performance; the CSO frequency would be reduced to between 7 and 3. Complete CSO elimination would require new storage of 41,000 m^3 in arrangement 2 and 51,000 m^3 in arrangement 3. A larger capacity was required by arrangement 3 because the two unconnected new tanks in this arrangement could not be fully utilized simultaneously. In general, the performance of arrangement 4 was very close to that of arrangement 3. For brevity, results of arrangement 4 are not presented.

It will be noted that, as CSO frequency decreased, a larger increment in capacity was required to reduce the frequency further by 1. Diminution in control efficiency as the degree of control increased, in fact, occurred also in CSO volumes and pollutant loads.

Estimated CSO volumes against new storage are shown in Figure 7.3. Unlike CSO frequency, however, CSO volume decreased with each increment of new storage capacity. This trend of CSO volume reduction would be particularly useful for fecal coliform reduction, because fecal coliform load was proportional to CSO volume. For complete CSO elimination, the seasonal volume of overflow stored and subsequently returned to the Humber WPCP was 334,000 m^3. It was less than 4% of the total flow treated by the WPCP during all the 26 events and was about 1/2% of the total volume treated in

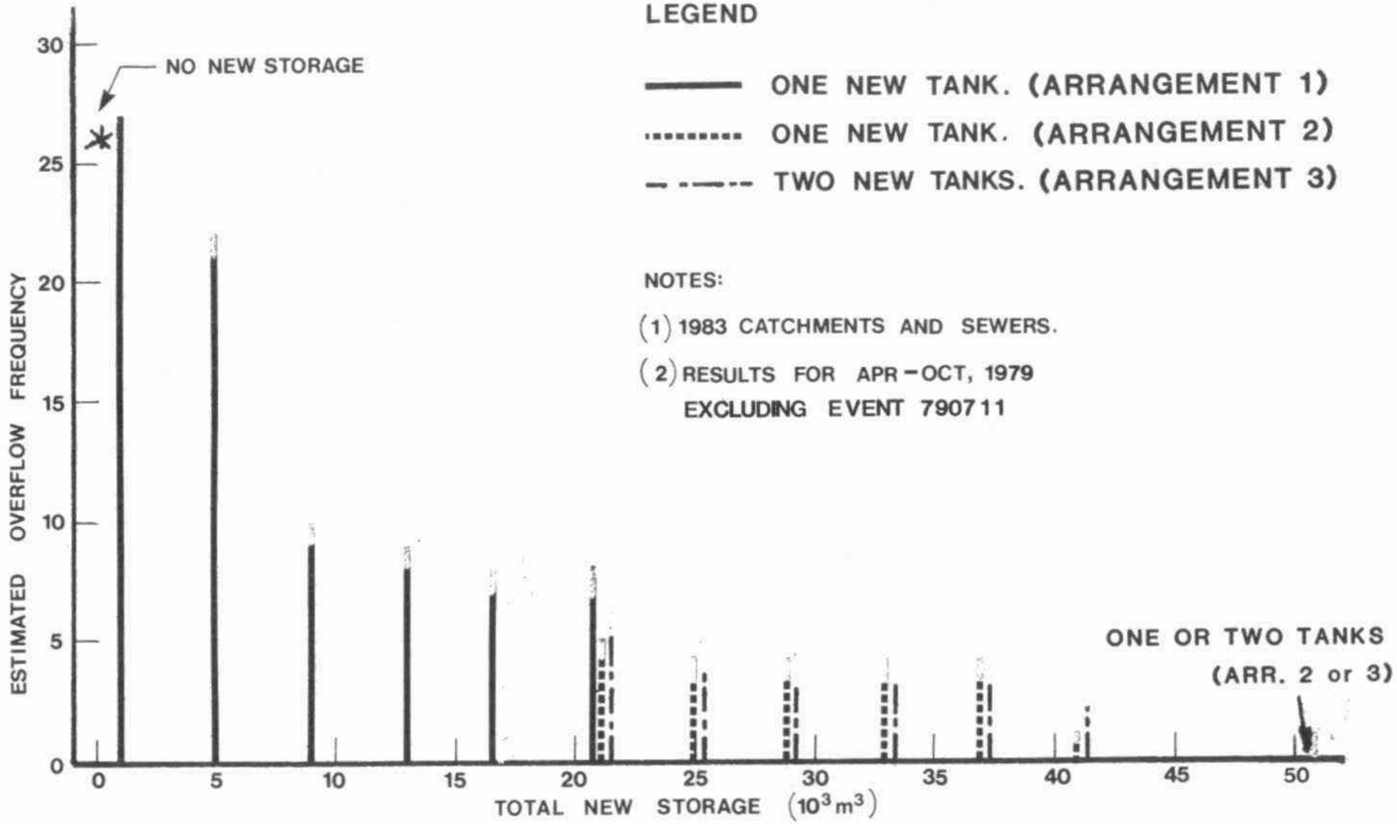


FIGURE 7.2: CONTROL BY STORAGE (BLACK CREEK GROUP)—OVERFLOW FREQUENCY DIAGRAM

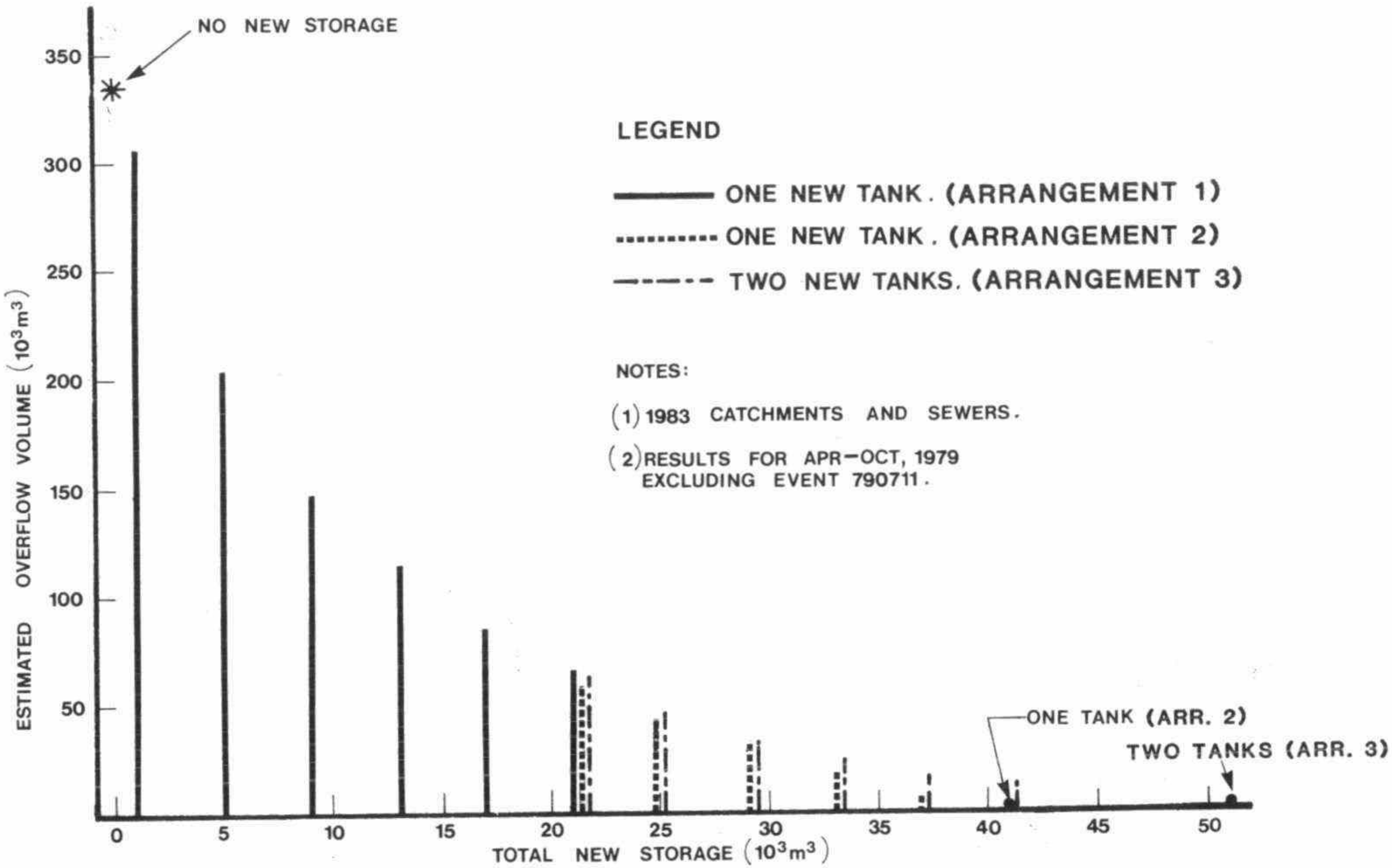


FIGURE 7.3: CONTROL BY STORAGE (BLACK CREEK GROUP)—OVERFLOW VOLUME DIAGRAM

the season. The corresponding increase in treatment cost should be insignificant.

As an illustration of CSO pollutant load reduction, SS reductions are summarized in Table 7.1. The impacts of CSO pollutant load reductions on the qualities of the receiving waters would be studied by a separate TAWMS project.

7.2.2 Engineering Considerations and Costs of Scheme 1

From an operational point of view, arrangement 2 is preferable to arrangement 3 because the facilities in arrangement 2 are more centralized. Arrangement 2, however, has 2 apparent disadvantages. First, the connector sewer between the existing tank and the new tank will be laid in a built-up area. Second, the performance of arrangement 2 depends much on the capacity of the long connector sewer (1.5 km in length) between the existing and new tanks. In a season other than the study season, the precipitation intensities of some storms may exceed the intensities of the storms in the study season even though the precipitation depths of the storms in the two seasons are comparable. Precipitation intensities often vary much more widely than precipitation depths. When the prevailing precipitation intensity exceeds the design intensity, it will be difficult for the connector sewer to accommodate the excessive flow peaks, because the frictional resistance to flow in a long pipeline is proportionally large. The consequence would be that CSO could occur at the emergency opening more often than expected.

The order of costs for complete elimination of CSO is shown below:

Tank Arrangement 2	Order of Cost
Enclosed tank, 41,000 m^3 near Site 3	\$2.2 million
Connector sewer, 2 m dia x 1.5 km	1.7
Total	3.9
Tank Arrangement 3	
Enclosed tank, 16,000 m^3 near Hyde Avenue	\$0.9 million
Enclosed tank, 35,000 m^3 near Site 3	1.9
Total	2.8

Arrangement 3 is therefore more preferable.

TABLE 7.1

ESTIMATED SS OVERFLOW LOADS
CONTROL BY STORAGE (1)

New Storage Capacity (m3)	SS Overflow Load (kg)	SS Overflow Load as % of No Control
0	63,000	100 %
1,000	47,000	75
5,000	38,000	60
9,000	31,000	49
13,000	24,000	38
17,000	18,000	29
21,000	13,000	21
25,000	10,000	16
37,000	1,770	3
41,000	0	0

Note :

(1) Catchments and sewers as in 1983.
Totals for April - October 1979,
excluding 790711

It is recommended that future feasibility study and design of any CSO storage tank (in this scheme or the other schemes) should consider the matters mentioned below:

An enclosed tank should be force-ventilated.

The inlets and emergency overflow device of a tank should be designed to ensure that the influent sewers will not be surcharged when the tank is filled to its high-water level and that the overflow device will not be flooded by the Black Creek at high flow. Preferably, the tank should be able to return the detained flow to the Black Creek STS by gravity. If this is not feasible, return flow pumping should be provided.

Since the capacity of the tank will be exceeded in a storm that is more intense than the design storm, consideration should be given to the need of disinfection of the overflow to the Black Creek.

An access road for heavy maintenance machinery to reach the tank is essential. Adequate facilities for maintenance and operation of the tank should be provided at the tank site. Flow monitoring instrumentation at the tank site should be provided for monitoring performance of the tank.

7.3 CSO Control Scheme 2: Resetting of Regulators

This scheme increased combined sewage flow to the Humber WPCP during wet weather by resetting the regulators. The concurrent use of new storage to intercept overflow at the regulators was also examined. The new storage tanks were assumed to be arranged in the same manner as in the previous scheme.

Scheme 2 would not be feasible if the peak primary treatment capacity of the WPCP should be revised from 11.8 m^3/s to 8.9 m^3/s as mentioned in Section 3.4. Scheme 2 presented in this section assumed the WPCP to have the original primary treatment capacity of 11.8 m^3/s.

The maximum allowable total increase in combined sewage flow to the WPCP was set at 2.0 m^3/s which is about 17% of the Humber WPCP peak capacity. The amount of increased flow was determined subject to 2 constraints: that the peak primary treatment capacity of the WPCP should not exceed 11.8 m^3/s and that sewage levels in the sewers upstream of the regulators would not be raised. Primary effluent in excess of the secondary treatment capacity would bypass the secondary treatment process to the effluent outfall after chlorination.

The threshold capacities of the regulators, before and after resetting to accommodate the increased flow, are indicated below:

	Before Resetting	After Resetting	Increase
Site 3	1.64 m^3/s	2.64 m^3/s	1.00 m^3/s
Mt. Dennis	0.32	1.12	0.80
Rockcliffe	0.47	0.67	0.20
Berry Road	10.20	No change	0

A proportionally higher increase was given to the Mt. Dennis regulator because this regulator discharged a larger percentage of its combined sewage to the Black Creek than did the other regulators.

The capacity of the existing Black Creek STS between the Humber STS and Site 3 is just sufficient for the maximum peak flow of 4.3 m^3/s of the base case. Scheme 2 assumed a duplication of this section of the Black Creek STS to provide the additional capacity of 2.0 m^3/s to prevent surcharge of the existing sewers upstream of the regulators.

7.3.1 Performance of Scheme 2

If resetting of regulators was used alone, CSO frequency would be reduced from 26 to 19 (Figure 7.4) and CSO volume from 334,000 m^3 to 92,000 m^3 (Figure 7.5) in a season. The reason for the marked difference in CSO frequency and volume performances was that resetting of regulators was able to reduce the overflow volumes

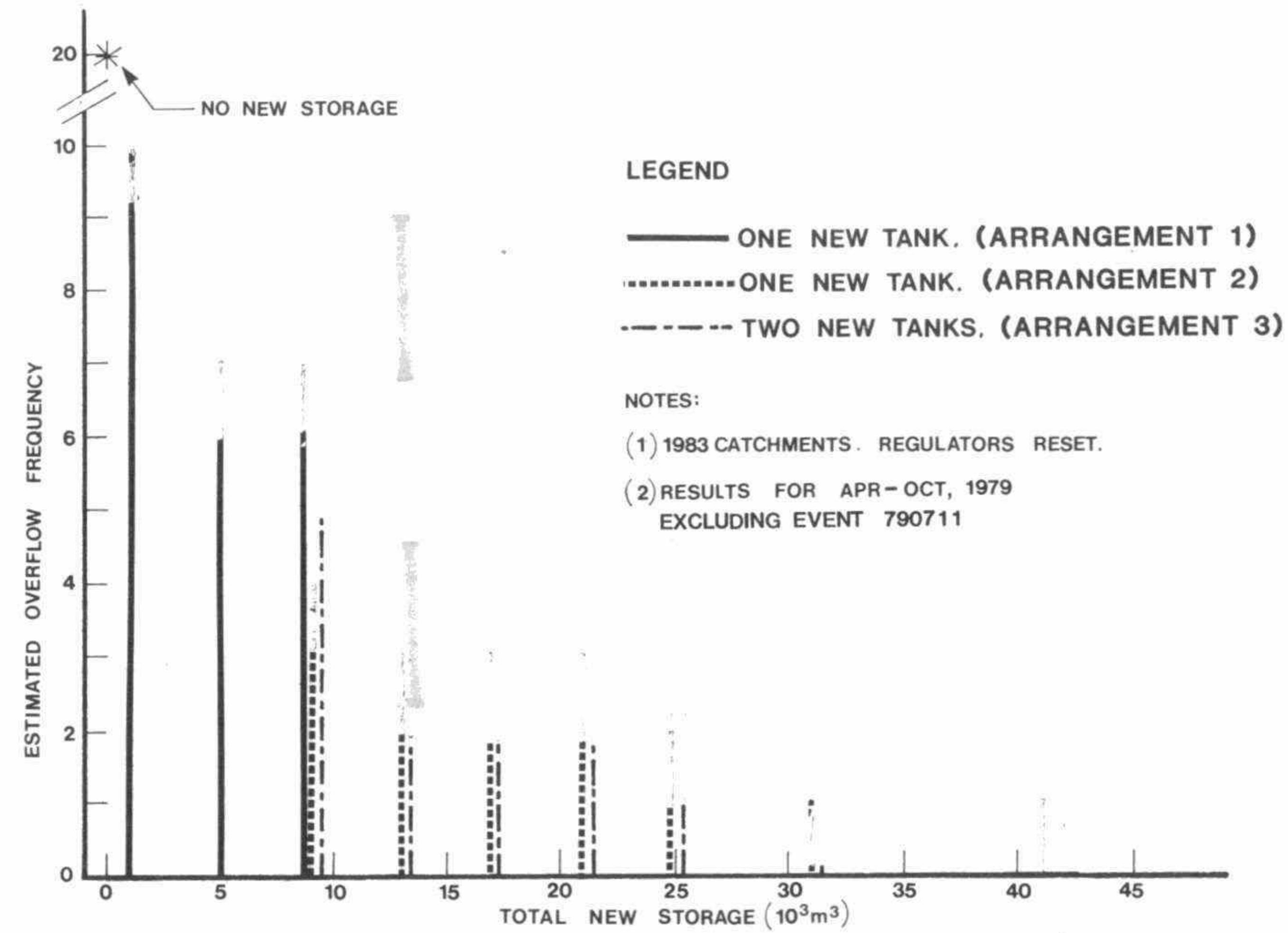


FIGURE 7.4: CONTROL BY RESETTING REGULATORS AND STORAGE (BLACK CREEK GROUP) —OVERFLOW FREQUENCY DIAGRAM

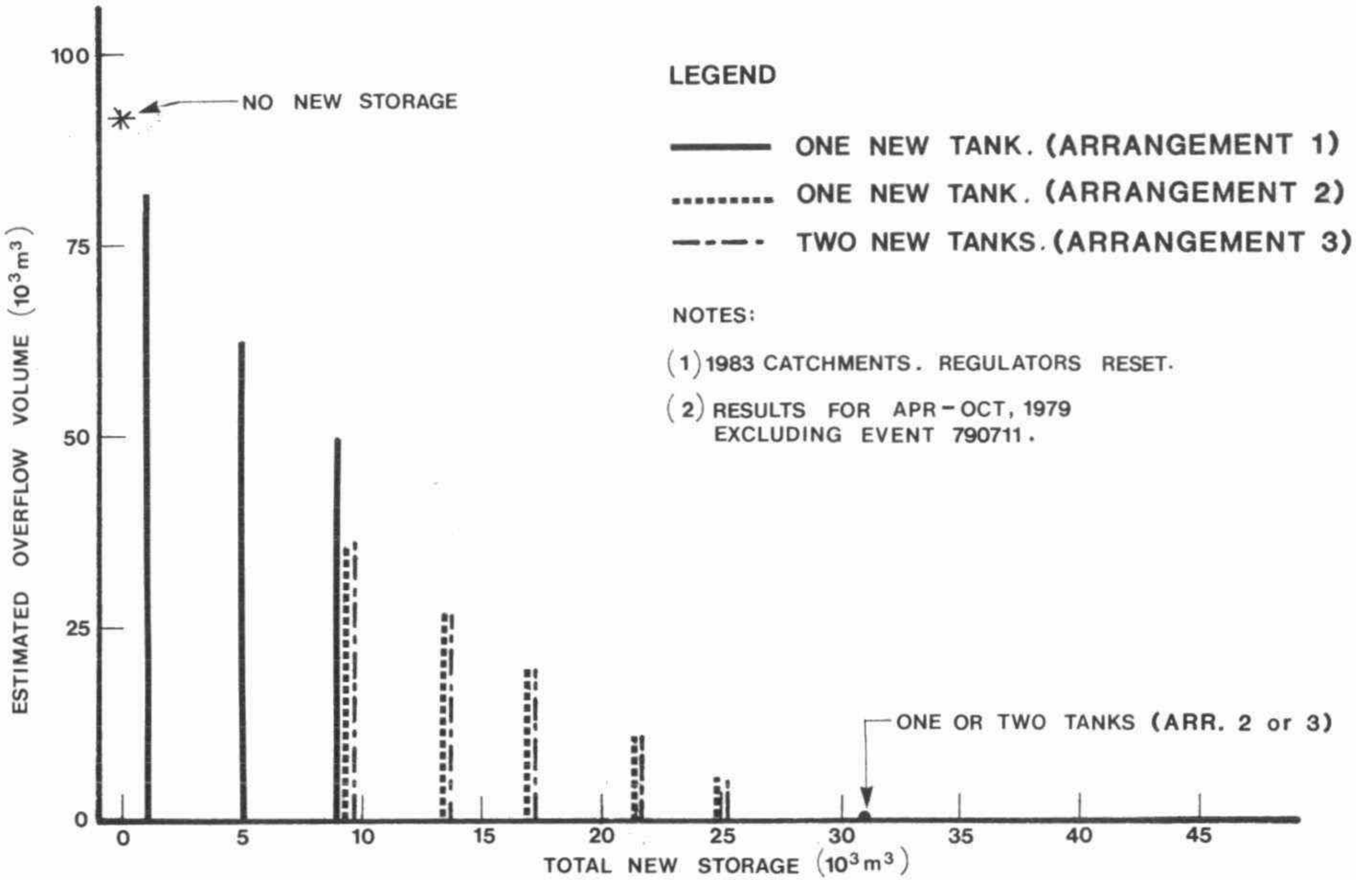


FIGURE 7.5: CONTROL BY RESETTING REGULATORS AND STORAGE (BLACK CREEK GROUP) —OVERFLOW VOLUME DIAGRAM

substantially but the reduction was not enough to completely eliminate a number of CSO occurrences. For CSO control beyond the degrees indicated above, other methods would be required to augment the resetting of regulators. New storage was used for this purpose.

Compared with Scheme 1, Scheme 2 required a lesser amount of new storage to achieve the same degree of control. This was expected. For example, to eliminate CSO completely, Scheme 2 would require only 31,000 m^3 new storage compared with 51,000 m^3 in Scheme 1. For complete CSO elimination using Scheme 2, the seasonal increase in combined sewage volume treated by the WPCP would be 334,000 m^3, the same as Scheme 1. In Scheme 2, however, 242,000 m^3 of this volume would receive primary treatment only, since it was brought to the WPCP during storms as the result of resetting the regulators. The remaining 92,000 m^3 would be sewage returned from storage to the WPCP after storms and would receive full treatment.

As an illustration of CSO pollutant load reductions, SS load reductions are summarized in Table 7.2. The resetting of regulators reduced the CSO SS load from 63,000 kg to 23,000 kg, a reduction of 63%.

7.3.2 The Berry Road Regulator

In Scheme 2, the Berry Road regulator overflowed for one hour in each of two events other than Event 790711. The hydrographs of the worse of these 2 events are shown in Figure 7.6. The overflow rate was only about 6% of the prevailing flow rate in the sewer. The CSO volume was about 4,000 m^3 in each event. Note that the overflow was less than the expected error in hydrologic modelling, and should be considered as an uncertain result.

It is suggested that provision of a detention tank to intercept the overflow of 4,000 m^3 should be withheld unless further flow monitoring confirms the need of this tank. If it is required, two alternative locations may be considered at the Berry Road regulator; or in the WPCP compound at the plant inlet. The choice will depend on detailed feasibility study.

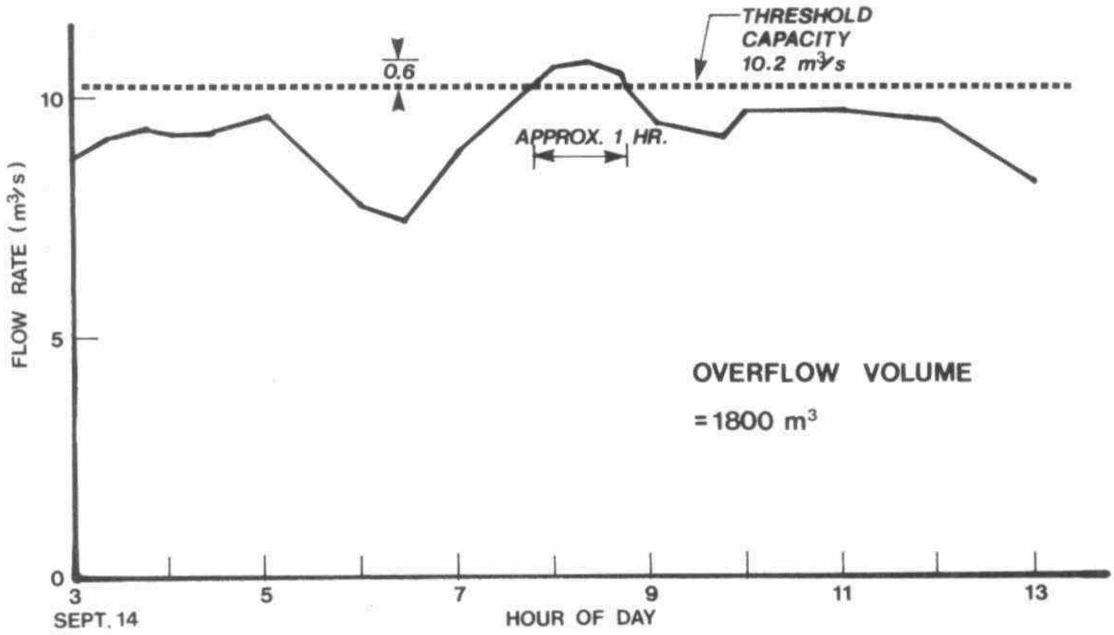
TABLE 7.2

ESTIMATED SS OVERFLOW LOADS
RESETTING REGULATORS (1)

In Combination With New Storage (m3)	SS Overflow Load (kg)	SS Overflow Load as % of No Control
0	23,000	37
1,000	20,000	32
5,000	15,000	24
9,000	10,000	16
13,000	8,000	13
17,000	5,650	9
21,000	3,470	6
25,000	1,800	3
37,000	0	0
41,000	0	0

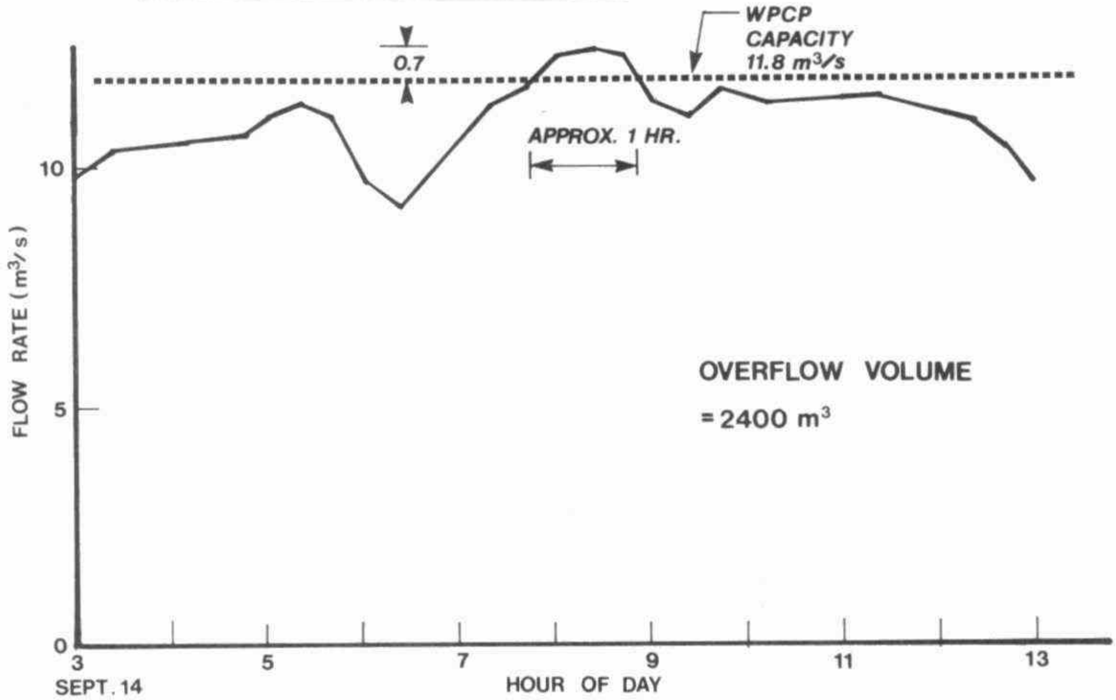
Note :

(1) Catchments as in 1983. Regulators reset.
Totals for April - October 1979, excluding 790711.



(A) *AT UPSTREAM END OF BERRY RD. REGULATOR*

Note: 1983 catchments. Regulators reset.



(B) *AT UPSTREAM END OF WPCP INLET*

FIGURE 7.6: HYDROGRAPHS OF EVENT 790913-14

7.3.3 Feasibility Considerations and Costs of Scheme 2

The order of costs of Scheme 2 for complete elimination of CSO is shown below:

	Order of Cost
Enclosed tank, 16,000 m^3 near Hyde Avenue	0.9 million
Enclosed tank, 15,000 m^3 near Site 3	0.9
Enclosed tank, 4,000 m^3, at Berry Road	0.4
Black Creek STS duplication, 1.2 m dia x 2.1 km	1.0
Special allowance	0.3
Total	3.5

The special allowance was made for two factors. About 500 m of the duplication sewer would lie across a golf course. Extra costs would probably be incurred for temporary reprovisioning work during sewer construction and for final rehabilitation of the disturbed golf course. As well, the duplication sewer would run along the Black Creek where soil conditions and ground water table could increase construction costs.

Scheme 2 would cost more than Scheme 1. In wet weather, Scheme 2 would stress the WPCP capacities to the limit. There would be no spare capacity to cope with any contingency or a possible future increase in flow due to urban redevelopment at a higher density than existing densities or due to an increase in water consumption.

In summary, Scheme 2 is less competitive than Scheme 1.

7.4 CSO Control Scheme 3: Stormwater Runoff Control

This scheme reduced CSO by controlling stormwater runoff in the combined sewer catchments or in the local combined sewers. A number of methods for controlling stormwater runoff are mentioned in the literature. The more promising ones, which were considered in Scheme 3, are listed below:

1. Detention of flow in local combined sewers;

2. Disconnecting roof leaders from combined sewers and infiltrating stormwater from the roofs into pervious surfaces around houses;

3. Combined sewer separation; and

4. Inlet restriction of stormwater at catchbasins.

Each of these methods is equivalent to reducing the size of the combined sewer catchment.

Reduction of combined sewer area was examined for 2 cases: 20% in Scheme 3A and 28% in Scheme 3B. The percentages of catchment reductions were selected arbitrarily, but the selection had been planned to obtain CSO control within a practical range of runoff control. If other degrees of CSO control are wanted, results may be obtained by interpolation of the available simulation results or by additional simulations.

The analysis approach was that the study model was first used to estimate CSO reductions resulted from the catchment reductions. Then the extent of each of the above 4 methods required for achieving the catchment reductions was estimated. Finally, an order of costs of control using these methods was determined and the relative merits of the methods were evaluated.

A breakdown of catchment reductions is shown in Table 7.3. The "Remainder" and the Kitchener sub-catchments of the Hillary catchment (Figure 3.1) were selected for control because basement flooding complaints arose most frequently in these sub-catchments. Larger percentages of catchment reduction were applied to the Mt. Dennis and Rockcliffe catchments to take advantage of their higher percentages of imperviousness.

In both Schemes 3A and 3B, the use of stormwater runoff control alone and in combination with new storage at regulators was studied.

TABLE 7.3

CATCHMENT REDUCTIONS IN CONTROL SCHEMES 3A AND 3B

Catchment	Area Before Reduction (ha)	Reduction in Scheme 3A			Reductions in Scheme 3B		
		Gross Area (%)	Gross Area (ha)	Impervious Area (ha)	Gross Area (%)	Gross Area (ha)	Impervious Area (ha)
Kitchener	230.4	0	0	0	20	46.1	12.1
"Remainder"	514.7	20	102.9	27.1	30	154.4	40.6
Keele	137.8	0	0	0	0	0	0
Mt. Dennis	167.0	40	66.8	32.7	40	66.8	32.7
Rockcliffe	196.1	40	78.4	26.3	40	78.4	26.3
Total	1,246.0	20	248.1	86.1	28	345.7	111.7

7.4.1 Performances of Schemes 3A and 3B

In Scheme 3A, if runoff control was used alone, i.e. without new storage at regulators, CSO frequency was not reduced at all (Figure 7.7) but CSO volume was reduced by 50% to 165,000 m^3 (Figure 7.8). The marked difference between CSO frequency and CSO volume performances indicated that although volume reduction was substantial, the reduction was not sufficient to eliminate CSO occurrence in any of the storm events. As mentioned earlier, volume reduction should be particularly useful for the reduction of CSO fecal coliform loads. To obtain CSO control beyond the results indicated above, new storage was used to intercept overflow from the regulators. The results were very similar to those of Scheme 2, as can be seen by comparing Figure 7.4 with Figure 7.7, and comparing Figure 7.5 with Figure 7.8. Complete CSO elimination would require new storage of 29,000 m^3 at the regulators.

As an illustration of reduction in CSO pollutant loads by Scheme 3A, SS loads are shown in Table 7.4. Again, the results were very similar to Scheme 2's.

Results of Scheme 3B are shown in Figures 7.9 and 7.10 and Table 7.4. Despite the larger reduction in catchment areas, the CSO reduction performances of Scheme 3B were not much better than the performances of Scheme 3A. Therefore, it may be expected that the effectiveness of runoff control in reducing CSO would diminish rapidly beyond the extent of catchment reduction in Scheme 3B.

In view of the above findings, only Scheme 3A was considered further for the application of runoff control methods to produce the catchment reduction effect. The results are discussed in subsections 7.4.2 through 7.4.5.

7.4.2 Scheme 3A(i): Detention of Flow in Local Combined Sewers

This runoff control method assumed that underground tanks were installed in local combined sewers to detain the stormwater runoff from the catchment area deleted in model simulation. The detained

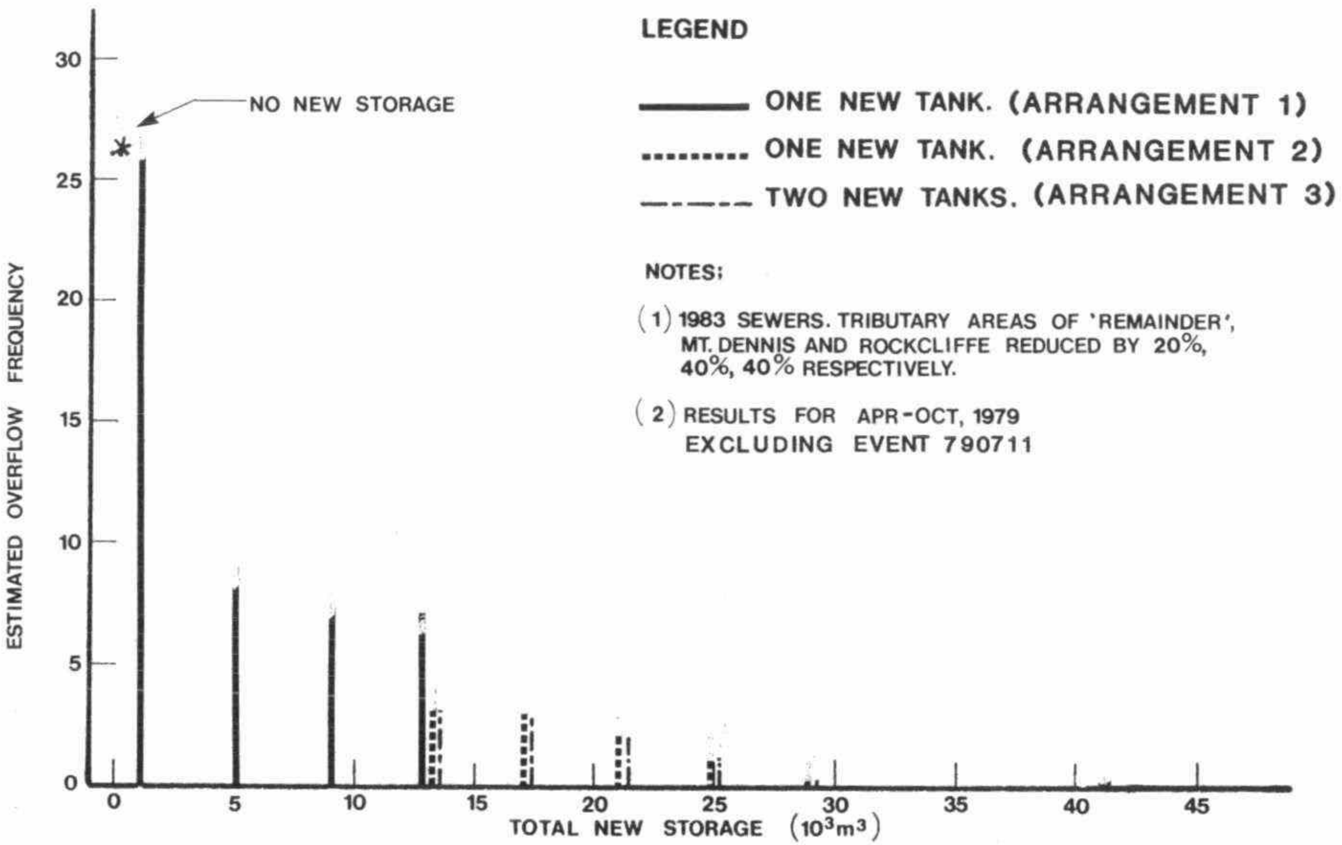


FIGURE 7.7: RUNOFF CONTROL AND STORAGE (A) — OVERFLOW FREQUENCY DIAGRAM

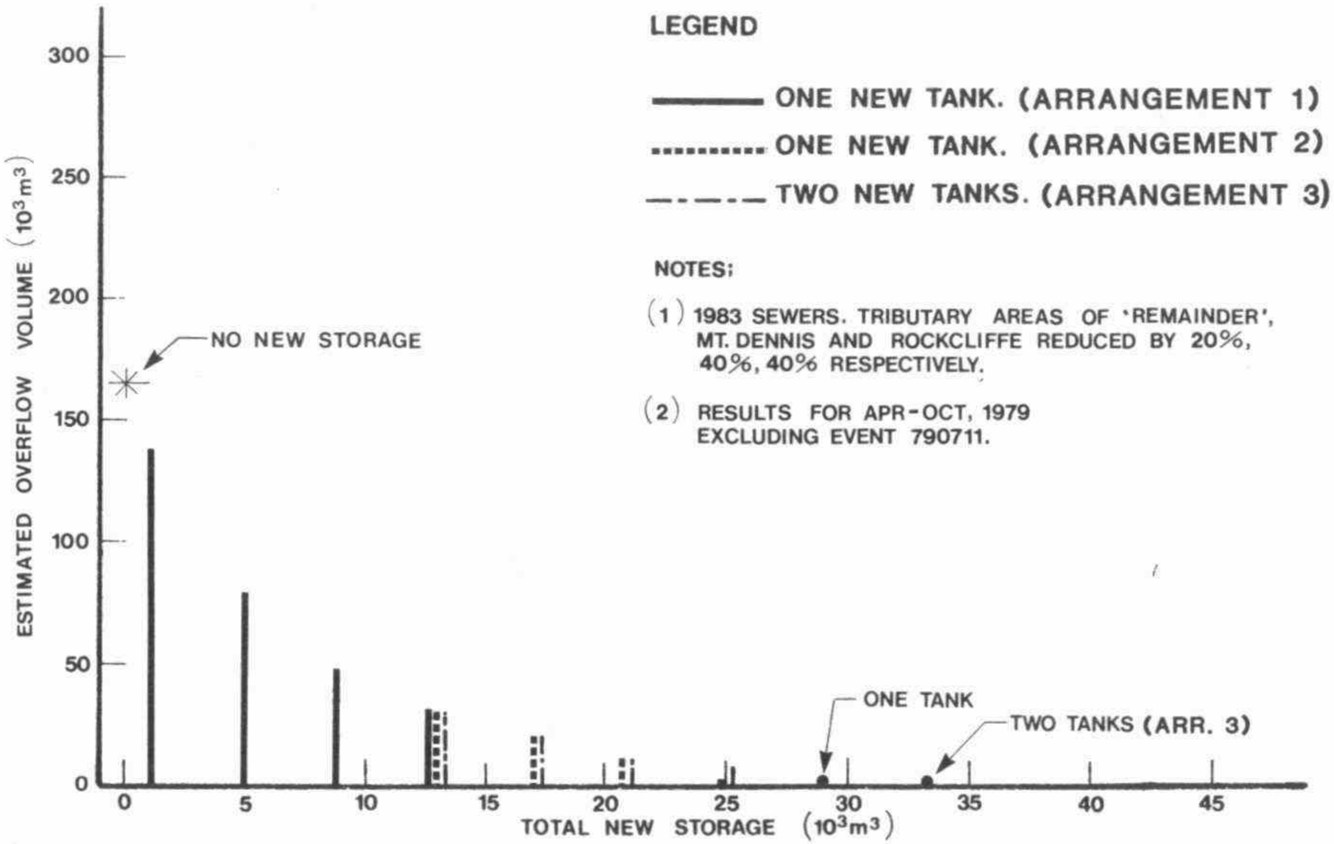


FIGURE 7.8 : RUNOFF CONTROL AND STORAGE (A) — OVERFLOW VOLUME DIAGRAM

TABLE 7.4

ESTIMATED SS OVERFLOW LOADS
RUNOFF CONTROL (1)

	Scheme 3A		Scheme 3B	
In Combination With New Storage (m3)	SS Overflow Load (kg)	SS Overflow Load as % of No Control	SS Overflow Load (kg)	SS Overflow Load as % of No Control
0	26,000	41	20,000	32 %
1,000	18,000	29	13,000	21
5,000	13,000	21	8,000	13
9,000	8,000	13	5,000	8
13,000	5,000	8	3,000	5
17,000	3,000	5	2,000	3
21,000	2,000	3	400	1
25,000	400	1	0	0
37,000	0	0	0	0

Note :

(1) Catchments assumed reduced. Regulators as in 1983.
Totals for April - October 1979, excluding 790711.

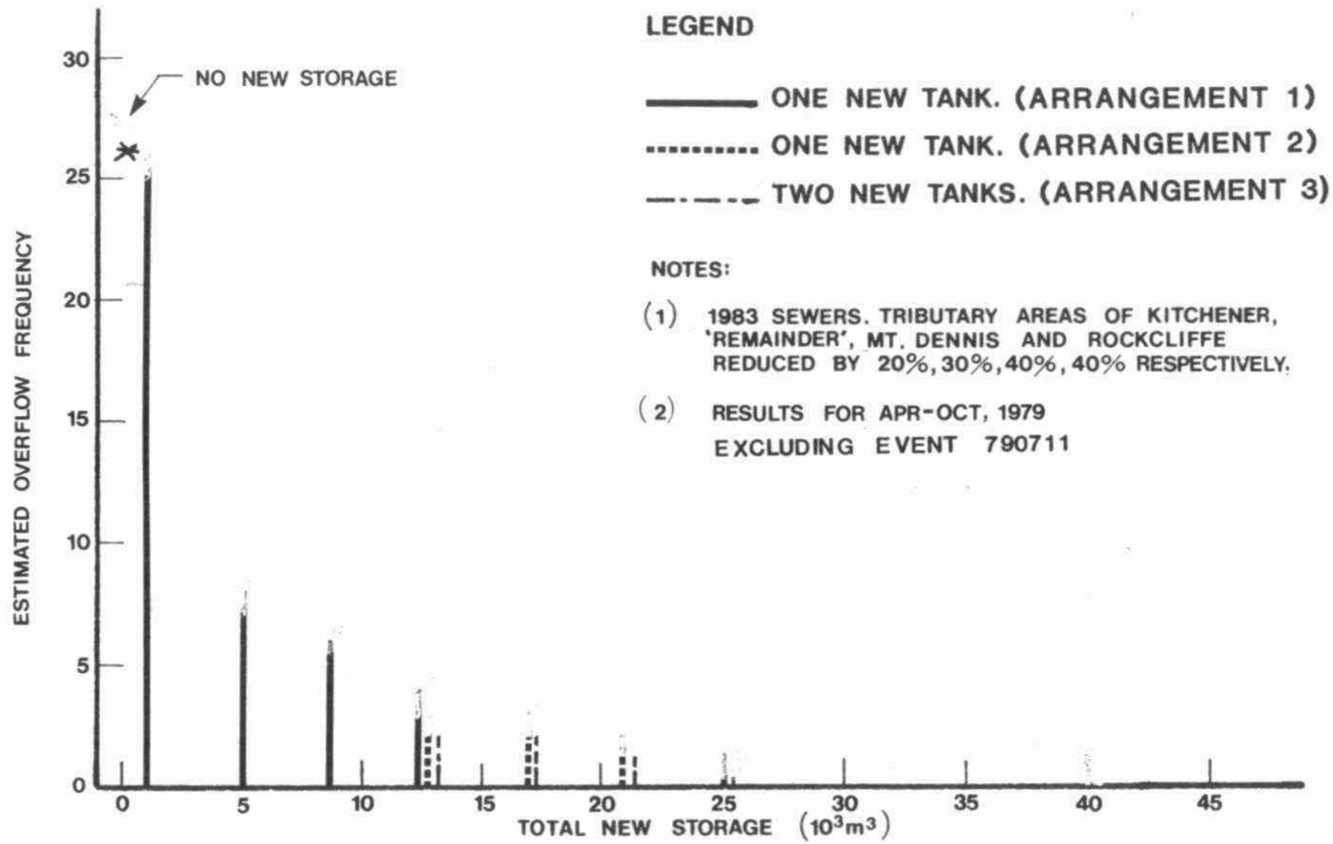


FIGURE 7.9 : RUNOFF CONTROL AND STORAGE (B) —OVERFLOW FREQUENCY DIAGRAM

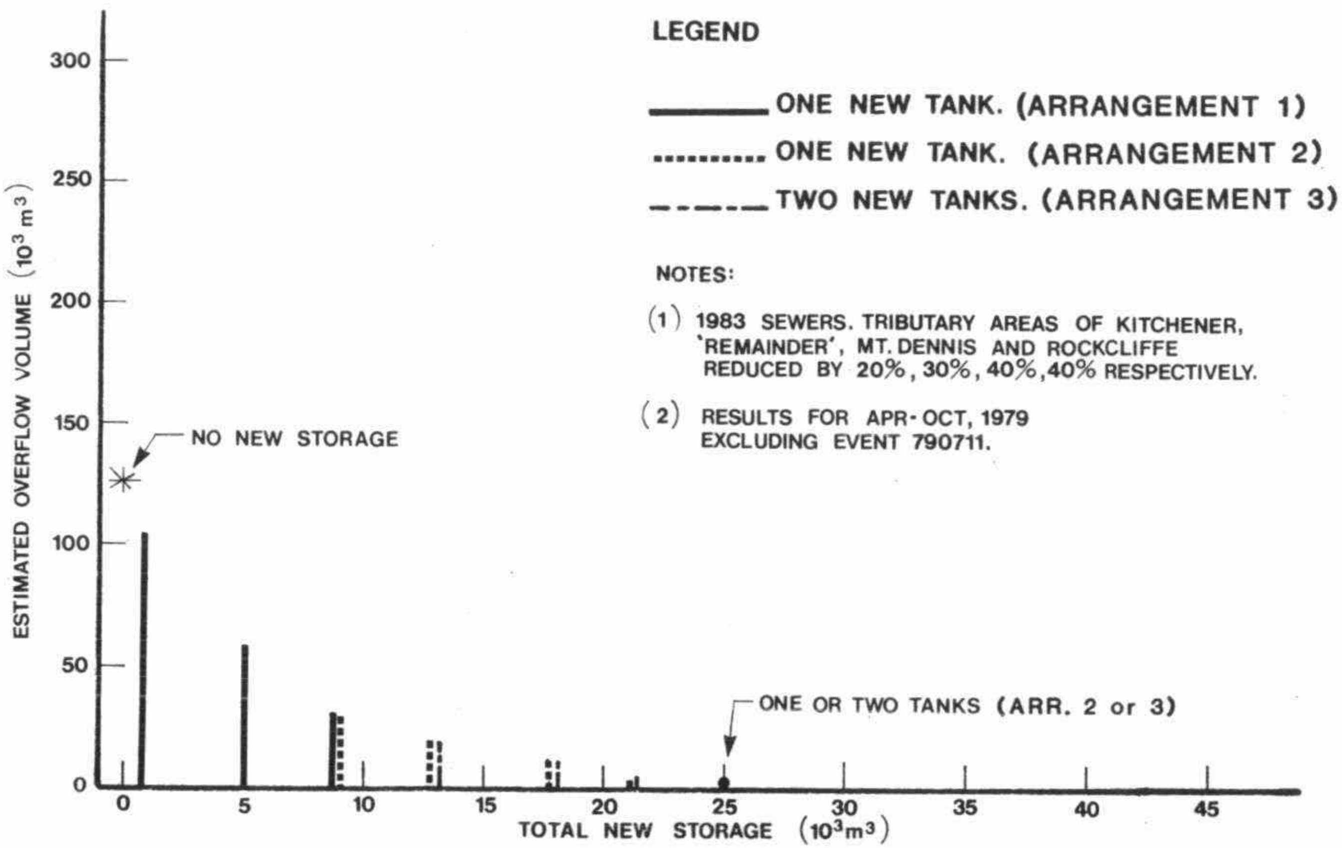


FIGURE 7.10 : RUNOFF CONTROL AND STORAGE (B) — OVERFLOW VOLUME DIAGRAM

runoff was assumed to be returned to the local combined sewers after the storm.

This method is a development in recent years and is gaining acceptance. So far, this method has been used for local flood control only and the City of York has used it in a limited scale for the same purpose. Typically, a local detention tank is built underground adjoining the sewer to be controlled and is made of a short length of a large-diameter pipe. A manhole provides access to the tank. Flow control valves are installed inside the manhole.

The total capacity of local detention tanks required was 14,882 m^3. It was estimated as the difference between the total flow volume (WPCP treated volume plus overflow volume) of Scheme 3A and the volume of the base case in the most critical event. Assuming each local detention tank to be a pipe of 2.5 m in diameter and 60 m in length, its capacity would be 295 m^3 and 50 tanks would be required. The order of cost for complete CSO elimination would be:

	Order of Cost
50 local tanks at $90,000	4.5 million
29,000 m^3 new storage near Site 3	1.6
Total	6.1

This estimate is more than twice Scheme 1's cost of $2.8 million. Except for high cost and the need for routine cleaning and maintenance of the tanks, this scheme does not have apparent serious disadvantages.

7.4.3 Scheme 3A(ii): Roof Leader Disconnection

It is well known that an increase in surface imperviousness of a catchment, such as due to urbanization, will increase stormwater runoff. It is often postulated that the converse of this phenomenon should be applied to reduce stormwater runoff in a developed catchment. One classic application, which was considered for Scheme 3A(ii), is disconnection of roof leaders of houses and

infiltrating the stormwater from roofs into pervious surfaces around the houses.

The area of impervious surfaces to be reduced was 86.1 ha. To obtain this reduction would require disconnection of roof leaders of 5,740 houses, assuming the average roof area to be 150 m^2 per house. The order of costs for complete CSO elimination would be:

	Order of Cost
Disconnecting 5,740 houses at $256 (Crozier, 1984)	$1.5 million
29,000 m^3 new storage near Site 3	1.6
Total	3.1

This estimate is conservative because the actual average roof area is probably smaller than 150 m^2 per house. In any case, Scheme 3A (ii) would cost more than Scheme 1 ($2.8 million).

This scheme is marginally feasible. The reliability of roof leader disconnection in this large scale is uncertain. Literature reporting on disconnection of existing roof leaders is scarce and reported cases seem to be concerned with basement flooding mitigation and not CSO control (Vatagoda, 1982). A 100% runoff reduction from the disconnected roofs cannot be expected for a number of reasons. For example, not all the receiving pervious surfaces are flat; some stormwater may run off from the sloping surfaces to sewers before the stormwater can infiltrate into the ground. Another possible reason is that the discharge from a roof leader may not spread on sufficient pervious surface for all the stormwater to infiltrate into the ground. In a roof leader disconnection demonstration project in the City of Stratford, the experimentor reported that no clear indication of flow reduction was observed (Crozier, 1984).

A potential problem of roof leader disconnection is seepage of surface water into basements due to poor surface drainage and flow blockage caused by snow.

Public acceptance of the scheme is uncertain and should be ascertained. The existing sewer bylaws of the City of York do not have specific provisions governing the disposal of stormwater from roofs, except for buildings erected after 1978 and buildings in a few designated streets where houses are prone to flooding.

In summary, this scheme should be considered for adoption only after the reliability and effectiveness of roof leader disconnection have been proven beyond doubt in a large scale pilot project.

7.4.4 Scheme 3A(iii): Combined Sewer Separation

The City of York is implementing a long-term combined sewer separation program developed in the late 1960's (Gore and Storrie Ltd., 1968). The program provides new local storm sewers at shallow depths (above existing sewers) to convey without surcharge 70% of the tributary storm drainage, and new trunk sewers at greater depths (below existing sewers) to convey 100% of the tributary storm drainage. The new trunk sewers are designed for a 2-year storm frequency using a City of Toronto (1965) standard. The storm is equivalent to a rainfall of intensity of 91.4 mm/hr for a duration of 8 minutes (Gore and Storrie Ltd, 1968).

Scheme 3A(iii) would require sewer separation in 248.1 ha (Table 7.3). The average cost of sewer separation in 1968 was $16,800/ha (Gore and Storrie Ltd, 1968). This cost updated to 1984 dollar value by a Canadian construction index of 297% (Fortin, 1985), would be $49,900/ha. Then the cost of the Scheme would be:

	Order of Cost
Sewer separation in 248.1 ha at $49,900	12.4 million
29,000 m^3 new storage near Site 3	1.6
Total	14.0

The above estimate is conservative because Scheme 3A(iii) assumed 100% separation but the original unit cost was based on 70% separation. In any case, the estimate already demonstrates sufficiently that the cost of sewer separation would be several times higher than the cost of Scheme 1.

Apart from high cost, sewer separation has 3 major disadvantages:

1. The sewer construction work will disrupt existing neighbourhoods extensively.

2. A sewer separation program typically takes many years to complete. Pending the final completion, new sewers laid during interim periods may not be able to function fully.

3. The runoff pollutants will still be transported by the new storm sewers to receiving waters. If the combined sewers are not separated and the CSO is intercepted, the pollutants will be treated at the WPCP. The estimated seasonal SS load discharged by the new storm sewers to the Black Creek in Scheme 3A(iii) was 67,000 kg, slightly more than the 63,000 kg CSO SS load before the sewer separation.

Sewer separation has one advantage in that the stormwater discharged from the storm sewers would contain much fewer fecal coliform bacteria than would CSO and the stormwater normally would not be contaminated with sanitary sewage. It should be noted, however, that this advantage will no longer exist if the CSO is eliminated by other schemes.

7.4.5 Scheme 3A(iv): Inlet Restriction of Stormwater at Catchbasins

This method controls the rate of stormwater runoff entering catchbasins from roads. The restriction is achieved by increasing the spacing of catchbasins in the case of a new development or by sealing some catchbasins in the case of an existing development. Restriction may be further augmented by installing an orifice in a catchbasin so that stormwater runoff entering a sewer via the catchbasin cannot exceed a pre-determined rate. The rejected stormwater runoff will use the road as its conduit. Consequently, this control method requires an outlet to be made available at the low spot of a road so that the runoff flowing down the road can

drain to a storm sewer, a watercourse or a detention facility, for example, a temporary pond in a park.

This method is a companion development of the local detention tank method. It has been used in some local sewer systems.

This method was not considered suitable for the combined sewer area of the present study because the road layout was not designed to integrate this use and the topography is generally unfavourable to the method. Therefore, this method was not evaluated further.

7.4.6 Conclusion of Runoff Control Methods

Except for the catchbasin inlet restriction method, all the runoff control methods are likely to be feasible for CSO control. The runoff control methods, however, all cost more than Scheme 1. In addition, the roof disconnection method and the sewer separation method have some disadvantages. Therefore, the runoff control methods should be placed in lower preference than Schemes 1 and 2.

8.0 INTEGRATING CSO CONTROL AND FLOOD PROTECTION MEASURES

8.1 Basic Design Requirements

Although the study of basement flooding mitigation in the combined sewer area is outside the scope of this CSO study, a cursory analysis of the feasibility of integrating measures for basement flooding mitigation into CSO control schemes was carried out and presented in this section.

To facilitate discussion, the conditions for the design of the local sewers and the CSO control schemes are repeated in Table 8.1. The term "frequency" used in relation to a storm deserves some comments. For one and the same storm, the frequency of the storm may assume different values depending on which slice of the storm is being referred to. For example, consider a synthetic storm. Based on City of Toronto (1965) standard used in the City of York's original sewer separation program (Gore and Storrie Ltd., 1968), a 2-year synthetic storm has an intensity of 68.5 mm/hr if the storm duration is 15 minutes, but an intensity of 91.4 mm/hr if the duration is 8 minutes. Therefore, the storm intensity used in design can be varied substantially merely by varying the assumed storm duration. Consider a synthetic storm again from another point of view. A storm of a given amount of precipitation, say 20 mm, has a 3-year frequency if the storm duration is 15 minutes, but a 10-year frequency if the duration is 8 minutes. It is obvious that mentioning the frequency of a storm without specifying which slice of the storm is being referred to is ambiguous. This explains why the single real storm shown in Table 8.1(B) has different recurrence intervals. (Recurrence interval is the reciprocal of frequency).

One more preliminary point should be noted. Flooding arises when the rate of sewage flow exceeds the sewer capacity. The length of time for which the sewer capacity is exceeded has no relation to the occurrence of flooding. On the other hand, CSO control is the containment of a flow volume which is the product of the overflow rate and the overflow duration. In other words, the design conditions to be satisfied for flood protection and CSO control are different.

TABLE 8.1

BASIC DESIGN CRITERIA OF SEWERS AND CSO CONTROL

(A) Design of City of York Sewers (1)

	Design Synthetic Storm			
	Reccurrence Interval (year)	Duration (minute)	Intensity (mm/hr)	Total Precip. (mm)
Existing Combined Sewers	1.5	15	63.5	12.5
New Storm Sewers	2.0	8	91.4	12.2

(B) CSO Control Schemes 1, 2, 3A and 3B

Characteristic of Real Storm (2)		Reccurrence Interval (year)
Average Intensity:	5.8 mm/hr	1.8
Max. Precip. in 1 hr:	16.2 mm	0.9
Max. Precip. in 2 hr:	16.4 mm	0.7
Total Event Precip.:	17.4 mm	<0.3

Notes:

(1) Gore and Storrie Ltd., 1968.
(2) Storm producing largest CSO volume in April-October, 1979, excluding Event 790711.

We shall now examine the feasibility of integrating flooding protection measures into CSO control measures. We shall consider two storms, a 2-year storm and a 5-year storm.

8.2 Two-Year Storm

An 8-minute synthetic storm was assumed, following the City of York's practice. The work required for basement flooding mitigation and the CSO control schemes are shown in the upper half of Table 8.2. As expected, CSO Control Schemes 1 and 2 would not offer flood mitigation. In order to eliminate basement flooding, and assuming that local detention tanks are to be used for this purpose, Schemes 1 and 2 would each require an augmentation by local storage of 14,000 m^3. This local storage would not benefit CSO control because it would not reduce the required capacity of the CSO control schemes.

CSO Control Scheme 3A(i) (Local Detention Tanks) and Scheme 3A(iii) (Sewer Separation) would provide the required flooding protection. It may be questioned, however, whether it is justified to incur the extra cost by using Scheme 3A(i) or 3A(iii) instead of Scheme 1 or 2 for the sake of increasing the basement flooding protection from a recurrence interval of 1.5 years to 2.0 years.

Whether CSO Control Scheme 3A(ii) (Roof Leader Disconnection) would offer flood protection would depend on the soil moisture prevailing at the time of the storm. The maximum (dry soil) infiltration capacity of black loam (a common type of top soil in lawns) is about 60 mm/hr (Linsley, 1975). If the soil is saturated, the stormwater from roof leaders will not be able to infiltrate into the soil and an augmentation by local storage of 14,000 m^3 would be required.

8.3 Five-Year Storm

Again, an 8-minute synthetic storm was assumed. The work required for satisfying CSO control and basement flood protection are shown in the lower half of Table 8.2. The situation in this case is clear

TABLE 8.2

WORK REQUIRED FOR CSO CONTROL AND FLOOD PROTECTION

Sewer Design Storm Considered	CSO Control Scheme Number		Will CSO Scheme Control CSO ?	Will CSO Scheme Provide Flood Protection ?	Local Storage Additional To CSO Control for Flood Protection (1)
2 - Year	1.	Overflow Detention	Yes. Flow to down-stream restricted by sewer capacity.	No	14,000 m^3
	2.	Resetting Regulators	Yes. Reason as above.	No	14,000 m^3
	3A.	Local Detention	Yes. Excess flow stored locally.	Yes	0
	3A.	Roof Dis-connection	Yes. Runoff not absorbed by soil flows overland.	Yes, if soil is dry.	0 if soil is dry. 14,000 m^3 if soil is saturated.
	3A.	Sewer Separation	Yes.	Yes	0
5 - Year	1.	Overflow Detention	Yes. As Scheme 1 above.	No	28,000 m^3
	2.	Resetting Regulators	Yes. As Scheme 2 above.	No	28,000 m^3
	3A.	Local Detention	Yes. Flow to down-stream restricted	No. 2-year storm only.	14,000 m^3
	3A.	Roof Dis-connection	Yes. As roof dis-connection above.	No. Soil capa-city exceeded.	14,000 m^3 if soil is dry. 28,000 m^3 if soil is saturated.
	3A.	Sewer Separation	Yes. Flow to down-stream restricted by sewer capacity.	No. New storm sewers designed for 2-year storm.	14,000 m^3

Note:
(1) For flood protection in runoff control areas assumed in Scheme 3A.

cut: basement flooding protection could be obtained only by control works in addition to the CSO control schemes. The function of the additional works was to reduce the flow rates to the capacities (for 1.5-year storm) of the existing sewers so that the sewers would not surcharge and basement flooding would not occur. The additional works would not reduce CSO because the flow put through by the additional works would not have a chance to flow through the sewers to the regulators if the additional works were not in place. The additional works in fact could increase CSO because that portion of the additional flow in excess of the threshold capacities of the regulators would overflow.

On the other hand, no works would be required for basement flooding protection if the duration of the 5-year storm was longer (say, 60 minutes) than the 8 minutes assumed in sewer design. In this case, the precipitation intensity would be 30.5 mm/hr only and the existing sewers would have sufficient capacities to transport the combined sewage. However, none of the CSO control schemes would have the capacity to contain the CSO from a precipitation of 30.5 mm falling in 1 hr because the precipitation was more than the 28.1 mm in 1 hr in Event 790711.

In conclusion, the measures required for CSO control and basement flooding protection in the City of York were distinctively different from each other and would not augment each other.

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APPENDICES

APPENDIX A1

CATCHMENT DATA

WASTEWATER PRODUCTION RATES (1)

Apartments

Reference: Table 3 for Years 1980-82

Avg. consumption = 586,581 gal/d = 2,666,597 l/d

Total population
= 2.3 persons/units (Table 2) x 3,475 units (Table 3)
= 7,993 persons

Avg. consumption = 2,666,597/7,993 = 333.6 lpcd

Offices

Reference: Table 4 for Years 1980-82

Land area: floor area = 569 ac: 13,644,000 sq ft
= 0.0417 ac/1,000 sq ft

Avg. consumption = 137 gal/0.0417 ac/d
= 36,905 l/ha/d

Note:

(1) All data from "North York: Summary of City-Wide Water Consumption Statistics (1976)-updated to 1982", except low/medium residential water consumption. The latter data were from Water Accounts Dept., City of Etobicoke, November 1983.

Shopping Centres

Total land area = 175.7 ac

Total avg. consumption = 290,997 gal/d = 18,604 l/ha/d

Commercial Consumption

Ratio of office land: shopping land = 569: 1,328 (Table 1)

Land Use Weighted Commercial Consumption
= 18,604 x 1,328 + 36,905 x 569 / (1,328 + 569) = 24,093 l/ha/d

Industrial Consumption

Avg. consumption on net area = 3,808 gal/ac/d

Vacant land = 26%

Avg. consumption on gross area = 3,808 x (100 - 26) / 100 gal/ac/d
= 31,653 l/ha/d

Low/Medium Residential Consumption

Avg. consumption per household in 124 days = 96,795 l

4.4 persons per household (North York Revised Table 2)

Avg. consumption = 96,800/4.4/124 = 177 lpcd

TABLE A 1.1

CATCHMENT DATA (Sheet 1 of 7) *
(A) COMBINED SEWER CATCHMENTS

Catchment I.D. No. (Gartner-Lee)	Catchment I.D. No. (CSO Model)	Catchment Name (If Any)	Land Use	Area (ha)	Population	WP (m3/d)	% Imperviousness	Overland Flow Length (m)	Road Length (km)
6.2.05	2531		1	135.1		2,564.1	18.9		
	2532		2	0.9		56.9	50.0		
(Excluding	2533	Kitchener	3	18.4		443.4	100.0		
Sanitary	2534	Ave	4	21.7		778.3	42.0		
Areas)	2535		5	22.2		6.8	5.0		
	2536		1	32.1			18.9		
		Sub-Total		230.4	14,656	3,849.4		2,304.0	23.9
	2541		1	301.8		5,727.4	18.9		
	2542		2	2.0		127.0	50.0		
	2543	"Remainder"	3	41.1		990.5	100.0		
	2544		4	48.5		1,738.5	42.0		
	2545		5	49.6		15.1	5.0		
	2546		1	71.7			18.9		
		Sub-Total		514.7	32,740	8,598.5		5,147.0	53.6
	2551		1	80.8		1,532.5	18.9		
	2552		2	0.5		34.0	50.0		
	2553	Keele	3	11.0		265.0	100.0		
	2554		4	13.0		465.2	42.0		
	2555		5	13.3		4.0	5.0		
	2556		1	19.2			18.9		
		Sub-Total		137.8	8,761	2,300.7		1,378.0	14.3

* See notes at end of table.

WP = Wastewater production rate.

TABLE A 1.1

CATCHMENT DATA (Sheet 2 of 7)
(A) COMBINED SEWER CATCHMENTS

Catchment I.D. No. (Gartner-Lee)	Catchment I.D. No. CSO) Model)	Catchment Name (If Any)	Land Use	Area (ha)	Population	WP (m3/d)	% Imperviousness	Overland Flow Length (m)	Road Length (km)
6.2.15	2151		1	101.5		1,364	33.0		
(Excluding	2152		2	4.0		203	50.0		
Sanitary	2153	Mount	3	14.5		349	100.0		
Areas)	2154	Dennis	4	34.0		1,079	90.0		
	2155		5	13.0		0	10.0		
		Sub-Total		167.0	8,314	2,995		2,451	17.4
6.2.18	2181		1	149.8		1,769	28.0		
	2184	Rockcliffe	4	24.5		776	95.0		
	2185		5	21.8		8	2.0		
		Sub-Total		196.1	9,994	2,553		1,900	16.1
Total of (A)				1,246.0	74,465	20,297		13,180	125.3

TABLE A 1.1

CATCHMENT DATA (Sheet 3 of 7)
(B) SANITARY SEWER CATCHMENTS

Catchment I.D. No. (Gartner-Lee)	Inlet I.D. No. (CSO Model)	Catchment Name (If Any)	Land Use	Area (ha)	Population	WP (m3/d)
3.1.07	42		1	268.4		1,969
			2	28.1		1,513
			3	38.3		923
			4	700.8		22,183
			5	173.0		60
		Sub-Total		1,208.6	15,655	26,588
3.1.08	21		1	718.2		6,271
			2	60.0		3,593
			3	56.8		1,368
			4	444.0		14,054
			5	405.8		103
		Sub-Total		1,684.8	46,190	25,389
3.1.09	14		1	510		2,990
			2	61		2,549
			3	14		347
			4	38		1,225
			5	319		103
		Sub-Total		944.5	24,527	7,214
3.1.10	2		1	483.1		3,611
			2	56.2		3,007
			3	28.0		646
			4	7.1		225
			5	238.4		78
		Sub-Total		812.8	29,402	7,567

TABLE A 1.1

CATCHMENT DATA (Sheet 4 of 7)
(B) SANITARY SEWER CATCHMENTS

Catchment I.D. No. (Gartner-Lee)	Inlet I.D. No. (CSO Model)	Catchment Name (If Any)	Land Use	Area (ha)	Population	WP (m3/d)
3.1.11	50		1	596.1		2,902
			2	43.7		1,432
			3	81.0		1,952
			4	302.9		9,588
			5	706.3		233
		Sub-Total		1,730.0	20,680	16,107
3.1.12	6 (3)		1	369.0		2,625
			2	27.8		1,452
			3	79.8		1,923
			4	351.1		11,113
			5	486.0		112
		Sub-Total		1,313.7	19,179	17,225
3.1.13	6		1	634.7		3,690
			2	20.3		885
			3	23.8		573
			5	75.8		26
		Sub-Total		754.6	23,499	5,174
3.1.14	94		1	400.8		2,346
			2	15.1		644
			3	47.7		1,149
			4	207.3		6,562
			5	258.9		86
		Sub-Total		929.8	15,185	10,787

TABLE A 1.1

CATCHMENT DATA (Sheet 5 of 7)
(B) SANITARY SEWER CATCHMENTS

Catchment I.D. No. (Gartner-Lee)	Inlet I.D. No. (CSO Model)	Catchment Name (If Any)	Land Use	Area (ha)	Population	WP (m3/d)
3.1.16	94		1	205.7		2,088
			2	38.9		2,952
			3	34.1		822
			4	225.0		7,122
			5	72.0		17
		Sub-Total		575.7	20,635	13,001
3.1.17	94		1	241.4		1,777
			2	21.8		1,083
			3	95.2		2,294
			4	492.4		15,586
			5	167.6		34
		Sub-Total		1,018.4	13,280	20,774
3.1.20 (4)						
4.1.02	120		1	990.7		13,057
			2	201.2		13,273
			3	84.1		2,026
			4	340.8		14,578
			5	868.5		259
		Sub-Total		2,485.5	113,509	39,403
4.1.03	150		1	747.1		6,486
			2	52.5		2,949
			3	46.4		1,118
			4	441.9		13,987
			5	258.5		60
		Sub-Total		1,546.4	45,472	24,600

TABLE A 1.1

CATCHMENT DATA (Sheet 6 of 7)
(B) SANITARY SEWER CATCHMENTS

Catchment I.D. No. (Gartner-Lee)	Inlet I.D. No. (CSO Model)	Catchment Name (If Any)	Land Use	Area (ha)	Population	WP (m3/d)
4.1.06	50		1	573.7		6,351
			2	55.3		3,445
			3	18.4		443
			4	666.8		21,106
			5	496.1		156
		Sub-Total		1,810.3	46,199	31,501
5.1.01	96		1	149.0		1,356
			2	12.4		852
			3	6.3		152
			4	7.5		321
			5	238.2		78
		Sub-Total		413.4	10,210	2,676
5.1.19	220		1	5.0		83
			3	0.9		22
			4	26.9		1,083
			5	6.2		
		Sub-Total		39.0	471	1,188
5.1.21	2		1	10.5		295
			3	4.0		96
			4	0.5		16
			5	0.5		0
		Sub-Total		15.5	1,669	407
6.1.04	10		1	39.9		238
			2	21.6		609
			3	1.0		24
			5	63.7		17
		Sub-Total		126.2	3,169	888

TABLE A 1.1

CATCHMENT DATA
(B) SANITARY SEWER CATCHMENTS (Sheet 7 of 7)

Catchment I.D. No. (Gartner-Lee)	Inlet I.D. No. (CSO Model)	Catchment Name (If Any)	Land Use	Area (Ha)	Population	WP (m3/d)
6.2.05	365		1	41.9		694
			2	10.3		548
			3	1.5		36
			4	11.0		471
			5	2.5		26
		Sub-Total		67.2	5,559	1,304
6.2.15	420		1	15.5		194
			2	1.0		51
		Sub-Total		16.5	1,250	245

Total of (B)	17,502.8	455,740	252,038
Total of (A)	1,246.0	74,465	20,297
TOTALS OF (A) + (B)	18,748.8	530,205	272,335

Notes :
(1) TWP = Theoretical waste water production rate.
(2) Land uses:
1 = Residual, low density.
2 = Residual, medium/high density.
3 = Commercial.
4 = Industrial.
5 = Open space and miscellaneous.
(3) Input of 17,280 m3/d from Mississauga not shown.
(4) Flow goes to Lakeview WPCP.

APPENDIX B1

INSTRUMENTATION FOR DATA COLLECTION

INSTRUMENTATION AND SAMPLE COLLECTION

ISCO 2500 series flow monitors were used in all stations except the inlet to the Hyde Ave tank and Station 5. These monitors are automatic computerized instrument recording the depths of flow, and were set to record at 5-minute intervals continuously. The depth sensors are small, streamlined submersible pressure transducers and were attached to the invert of the sewer or the overflow weir crest. The data collected were saved on a computer tape and subsequently transferred to micro-computer diskettes and processed by a micro-computer. Since there was no accessible location to monitor the inlet to the Hyde Ave tank, data for this location had to be obtained from the tank's permanent bubbler water level recorder. This instrument records on a weekly chart. To improve the time resolution of the time base of the recording, a second pressure signal recorder having a higher chart speed was installed. Despite this effort, however, the data for this station did not have as a high resolution as the ISCO stations' because the instrument read the depth of water in the tank (not in the sewer) and one graduation of the chart was 50 mm of the tank depth.

Due to persistent high flow in the sewer, flow data at Station 5 was measured with an N-Con surface tracking monitor, which is an analog instrument recording the flow depth on a paper chart continuously. The sewage surface is continuously tracked by a suspended probe linked by a mechanism to the recording pen.

The ISCO monitor worked satisfactorily for most of the time. On occasions it malfunctioned, mostly because of the seal failure on

the pressure transducers. The manufacturer replaced the faulty sensors under warranty.

For sample collection, ISCO model 2100 automatic samplers were used. Each sampler contained 24 sample bottles of 1-litre size. All of the monitoring gear is suitable for installation inside a manhole. Sample was collected via a suction tube and the suction pump was driven by battery. The sampler was actuated in two ways. On overflow sampling, float switches were used to signal the sampler when sewage passed over the weir. For sampling in the sewer line itself, the start time of the sampling cycle was pre-set and samples were taken at fixed time intervals. Samples and flow rates were time correlated based on known time setting.

In this project, DWF samples were 24-hour composite samples. For combined sewage, sequential samples were collected as data were needed for deriving a relationship of flow concentrations with flow rates (load rate curve) for later use with the simulation model. In a storm event, a number of sequential samples were first collected from Site 3. After the storm passed over, the event precipitation was examined to see if the samples should be analyzed by the laboratory. The aim of the sampling program was to collect data for 8 to 10 events and that the events should be approximately evenly distributed among the precipitation groups of 5 to 10 mm; 10 to 15 mm and over 15 mm. If analysis was justified, the hydrograph of the event was used to guide the selection of samples such that samples at the time of major crests and troughs of the hydrograph was picked. The selection was necessary to reduce the number of samples to be analyzed and yet to provide as much information about changes in pollutant concentrations as possible. About 6 to 8 samples were analyzed for a selected event.

Precipitation was measured with a tipping bucket rain guage and a standard guage, following Atmospheric Environment Services practices. The data were discretized into 5-minute intervals.

APPENDIX B2

FLOW QUANTITY AND QUALITY DATA

TABLE B 2.1

OBSERVED DRY WEATHER FLOW AT HILLARY AVE. SEWER CATCHMENT
(STATION I.D. No. 1) (Sheet 1 of 2)

(A) Dates of Data

	Week 1	Week 2	Week 3	Week 4
Mon	821115	830613	830620	No Data
Tue	821119	830510	830517	830524
Wed	821117	830511	830518	830601
Thu	821028	830428	830512	830602
Fri	821029	830429	830513	830527
Sat	821030	830528	830611	830618
Sun	821107	830605	830612	830619

(B) Average Daily Volume (m^3/d)

Week 1	Week 2	Week 3	Week 4	4-Week Avg.	Standard Deviation
25,970	27,775	26,768	27,270	26,946	770

(C) Ratios of Daily Flow Variations

	Week 1	Week 2	Week 3	Week 4	4-Week Avg	Standard Deviation
Mon	.89	1.02	1.00	No Data	1.01	0.07
Tue	.74	.99	.98	1.10	.95	0.15
Wed	.85	.99	.98	.99	.95	0.07
Thu	1.23	1.11	1.03	.98	1.09	0.11
Fri	1.27	.91	1.05	1.02	1.06	0.15
Sat	1.27	1.00	.99	.99	1.06	0.14
Sun	.74	.99	.96	.92	.90	0.11

TABLE B2.1 (Sheet 2 of 2)

(D) Ratios of Hourly Flow Variations

Hour	Week 1	Week 2	Week 3	Week 4	4-Week Avg	Standard Deviation
1	.86	.68	.69	.72	.74	.08
2	.72	.59	.61	.62	.64	.06
3	.63	.55	.57	.59	.59	.03
4	.60	.54	.55	.58	.57	.03
5	.57	.62	.58	.59	.59	.02
6	.59	.75	.72	.74	.70	.07
7	.71	.95	.98	.98	.91	.13
8	.96	1.15	1.21	1.20	1.13	.12
9	1.20	1.21	1.26	1.25	1.23	.03
10	1.25	1.23	1.28	1.26	1.26	.02
11	1.26	1.22	1.27	1.25	1.25	.02
12	1.26	1.18	1.22	1.20	1.22	.03
13	1.20	1.13	1.16	1.14	1.16	.03
14	1.16	1.08	1.11	1.10	1.11	.03
15	1.11	1.04	1.07	1.06	1.07	.03
16	1.07	1.02	1.05	1.06	1.05	.02
17	1.06	1.05	1.08	1.08	1.07	.02
18	1.10	1.58	1.17	1.14	1.25	.22
19	1.18	1.34	1.19	1.18	1.22	.08
20	1.22	1.12	1.14	1.13	1.15	.05
21	1.16	1.07	1.11	1.09	1.11	.04
22	1.10	1.06	1.09	1.10	1.09	.02
23	1.04	.99	1.03	1.03	1.02	.02
24	1.00	.85	0.88	0.89	0.91	.07

TABLE B 2.2

OBSERVED DRY WEATHER FLOW AT BLACK CREEK STS.
(STATION I.D. No. 5) (Sheet 1 of 2)

(A) Dates of Data

	Week 1	Week 2
Mon	830620	830711
Tue	830621	830712
Wed	830706	830713
Thu	830707	830714
Fri	830624	830708
Sat	830702	830813
Sun	830703	830710

(B) Average Daily Volume (m^3/d)

Week 1	Week 2
79,253	93,984

(C) Ratios of Daily Flow Variations

	Week 1	Week 2	2-Week Avg.
Mon	.95	.79	.87
Tue	.93	1.13	1.03
Wed	1.03	1.27	1.15
Thu	.94	1.28	1.11
Fri	.98	.80	.89
Sat	1.06	1.13	1.10
Sun	1.11	.61	.86

TABLE B2.2 (Sheet 2 of 2)

(D) Ratios of Hourly Flow Variations

Hour	Week 1	Week 2	2-Week Aug.
1	.84	.82	.83
2	.67	.65	.66
3	.54	.51	.53
4	.48	.47	.47
5	.46	.46	.46
6	.48	.49	.49
7	.66	.65	.66
8	.93	.89	.91
9	1.13	1.09	1.11
10	1.20	1.17	1.18
11	1.32	1.25	1.28
12	1.36	1.39	1.38
13	1.35	1.34	1.34
14	1.30	1.28	1.29
15	1.21	1.16	1.18
16	1.11	1.16	1.13
17	1.07	1.16	1.12
18	1.13	1.18	1.15
19	1.23	1.27	1.25
20	1.24	1.31	1.26
21	1.14	1.18	1.16
22	1.07	1.04	1.05
23	1.08	1.04	1.06
24	1.00	1.00	1.00

TABLE B2.3 (Sheet 1 of 2)

OBSERVED DRY WEATHER FLOW QUALITIES

Sampling Date	BOD5	RSP	PPUT	PPO4FR	CUUT	PBUT	ZNUT	Sampling Date	CDUT* Conc. At Mt. Dennis/ Rockcliffe	
	(Concentration (mg/l) at Hillary Ave. Catchment)									
821103	218.0	160.0	3.65	1.78	0.14	0.09	0.31	830216	0.01	(Rock)
821105	137.0	154.0	1.45	0.86	0.07	0.08	0.33	830223	0.01	(Rock)
821106	400.0	330.0	6.20	3.70	0.47	0.07	0.47	830224	0.02	(MT.D)
821109	265.0	299.0	6.50	3.35	0.15	0.08	0.25	830224	0.01	(Rock)
821110	187.0	180.0	4.45	2.34	0.12	0.03	0.12	830228	0.01	(MT.D)
821117	199.0	160.0	4.47	2.32	0.13	0.03	0.16	830301	0.01	(MT.D)
821118	200.0	237.0	5.10	2.54	0.10	0.04	0.19	830301	0.03	(MT.D)
821120	230.0	347.0	6.25	3.38	0.17	0.08	0.19	830303	0.02	(MT.D)
821121	184.0	421.0	12.20	2.40	0.21	0.13	0.26	830303	0.01	(Rock)
821122	135.0	249.0	4.25	2.22	0.09	0.04	0.13	830304	0.02	(MT.D)
821123	143.0	162.0	5.15	1.70	0.09	0.03	0.23	830304	0.01	(Rock)
Average	208.9	245.4	5.42	2.42	0.16	0.06	0.24		0.01	
Standard Deviation	75.1	92.4	2.66	0.83	0.11	0.03	0.10		0.007	

Notes:
RSP = Suspended Solids
CUUT = Copper
CDUT = Cadmium in mg/l
ZNUT = Zinc
PBUT = Lead
CDUT* Conc. in mg/l
PPO4FR = Filtered, Reactivi Phosphorous
PPUT = Total Phosphorous

TABLE B2.3 (Sheet 2 of 2)

OBSERVED DRY WEATHER FLOW QUALITIES

Sampling Date	BOD5	RSP	PPUT	PPO4FR	CUUT	PBUT	ZNUT	Sampling Date	CDUT* Conc. At Mt. Dennis/ Rockcliffe	
	(Concentration (mg/l) at Black Creek Trunk Sewer)									
821102	275.0	151.0	5.00	2.38	0.43	0.11	0.13	830216	0.01	(Rock)
821103	208.0	155.0	3.80	1.76	0.48	0.06	0.23	830223	0.01	(Rock)
821105	186.0	114.0	1.62	1.58	0.32	0.06	0.12	830224	0.02	(MT.D)
821106	240.0	150.0	14.80	1.40	0.36	0.04	0.16	830224	0.01	(Rock)
821107	157.0	153.0	3.45	1.50	0.18	0.02	0.11	830228	0.01	(MT.D)
821108	280.0	303.0	4.62	2.84	0.28	0.04	0.15	830301	0.01	(MT.D)
821109	177.0	177.0	4.50	4.40	0.50	0.05	0.20	830301	0.03	(MT.D)
821117	234.0	175.0	6.10	2.32	0.64	0.03	0.22	830303	0.02	(MT.D)
821118	230.0	156.0	6.20	2.38	0.47	0.03	0.15	830303	0.01	(Rock)
821119	143.0	145.0	5.50	2.62	0.37	0.02	0.15	830304	0.02	(MT.D)
821120	134.0	180.0	6.10	3.96	0.34	0.02	0.14	830304	0.01	(Rock)
821121	150.0	217.0	5.45	3.38	0.20	0.03				
821122	127.0	190.0	5.75	2.84	0.18	0.03	0.14			
821123	146.0	168.0	6.25	2.76	0.31	0.05	0.28			
Average	191.9	173.8	5.66	2.58	0.37	0.04	0.17		0.01	
Standard Deviation	52.4	44.3	2.94	0.89	0.13	0.02	0.05		0.007	

Notes:
RSP = Suspended Solids
CUUT = Copper
CDUT = Cadmium in mg/l
ZNUT = Zinc
PBUT = Lead
PPO4FR = Filtered, Reactivi Phosphorous
PPUT = Total Phosphorous

HILLARY COMBINED SEWER

OBSERVED TOTAL FLOW

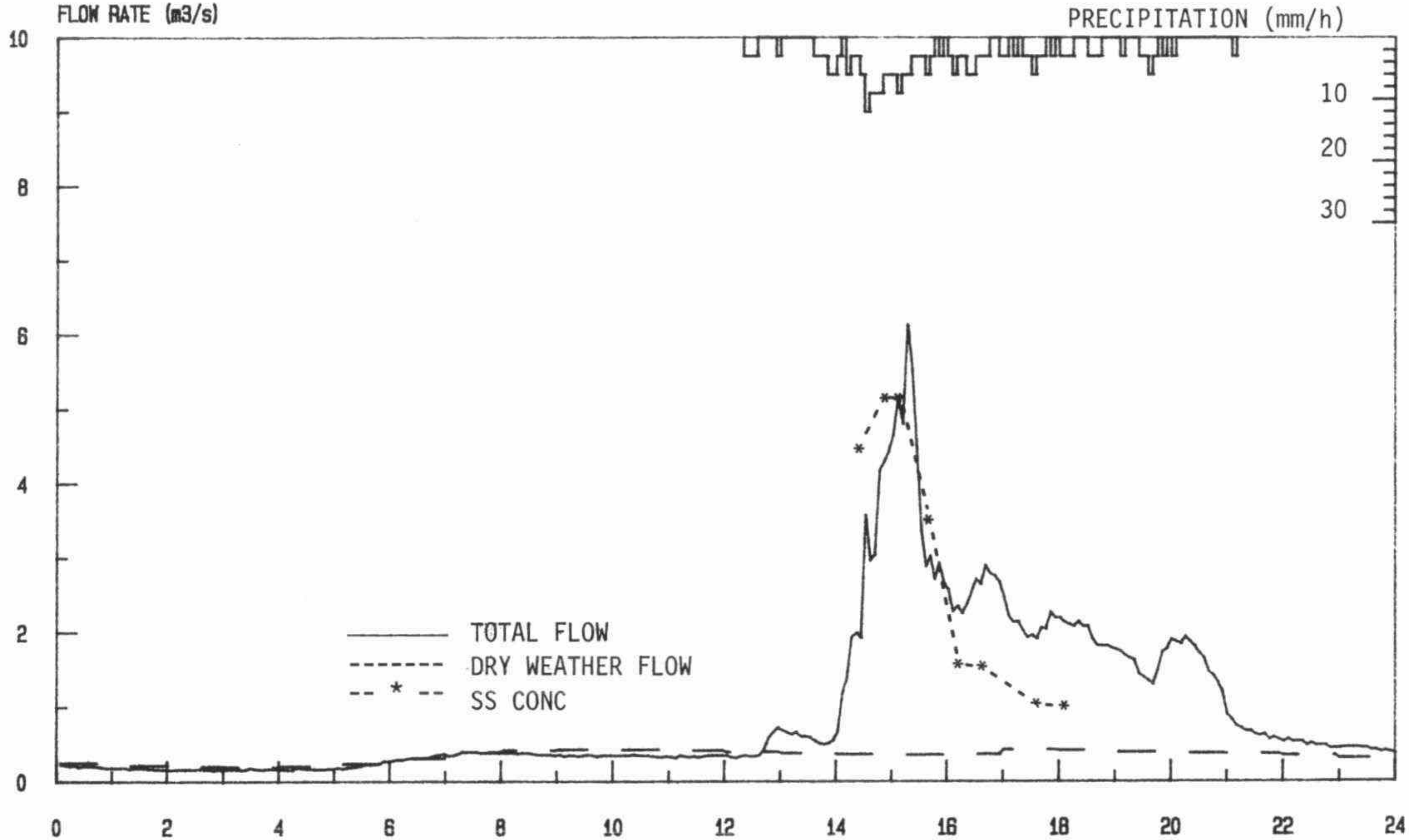


FIGURE B2.1

MAY 19, 1983

HILLARY COMBINED SEWER

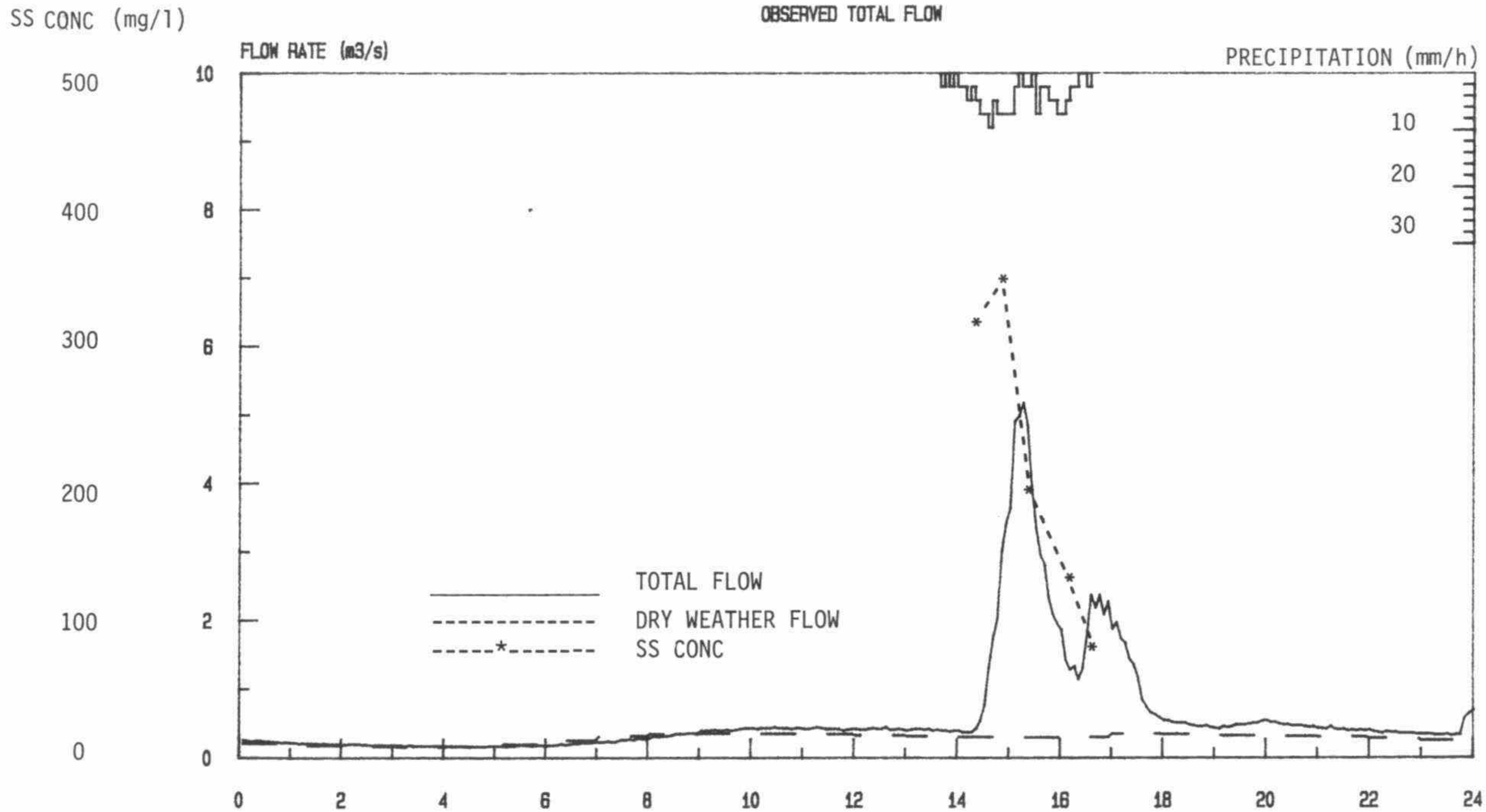


FIGURE B2.2

MAY 29, 1983

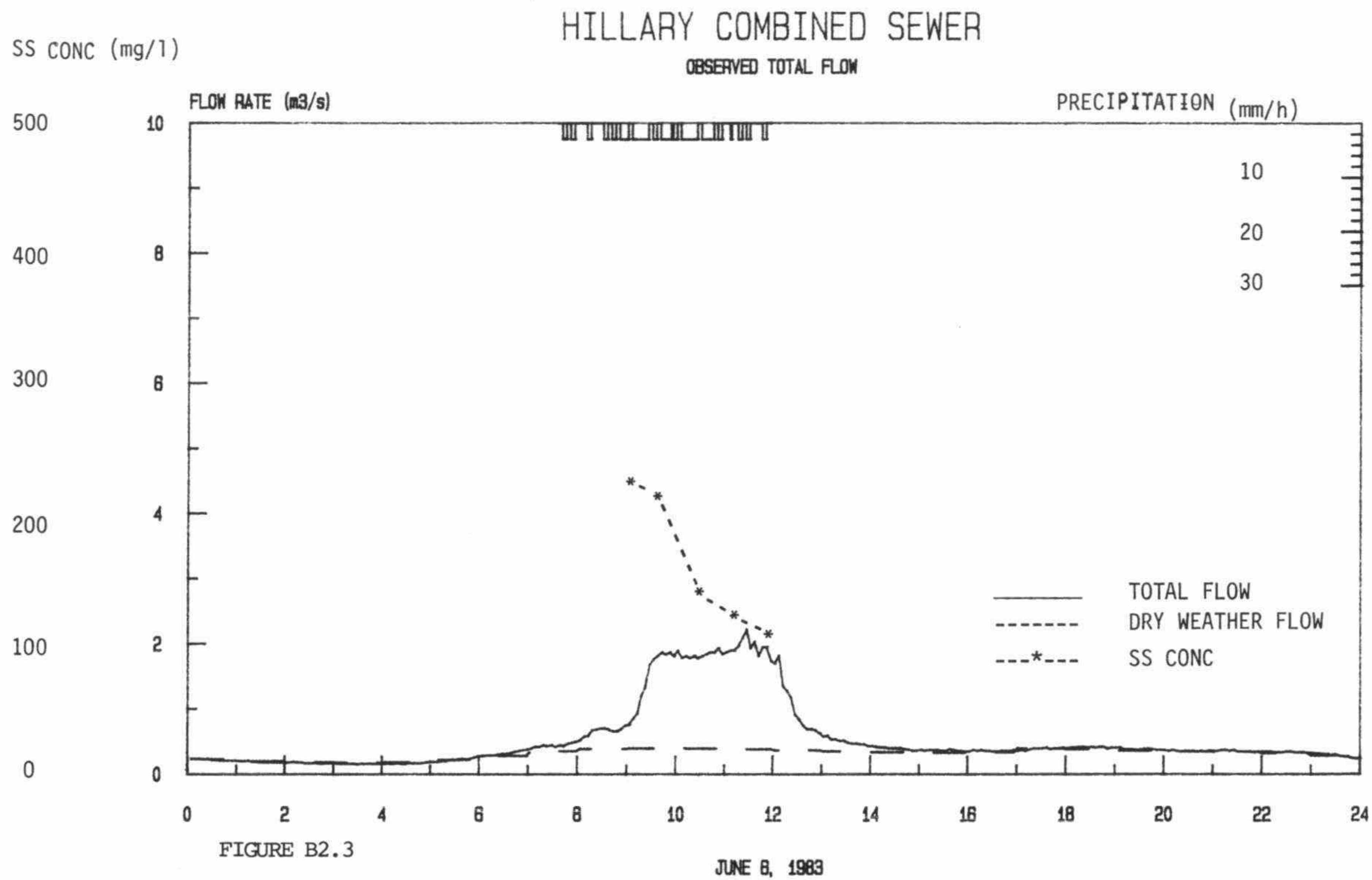

HILLARY COMBINED SEWER
OBSERVED TOTAL FLOW
SS CONC (mg/l)
FLOW RATE (m3/s)
PRECIPITATION (mm/h)
500
400
300
200
100
0
10
8
6
4
2
0
10
20
30
TOTAL FLOW
DRY WEATHER FLOW
SS CONC
0
2
4
6
8
10
12
14
16
18
20
22
24
JUNE 8, 1983

FIGURE B2.3

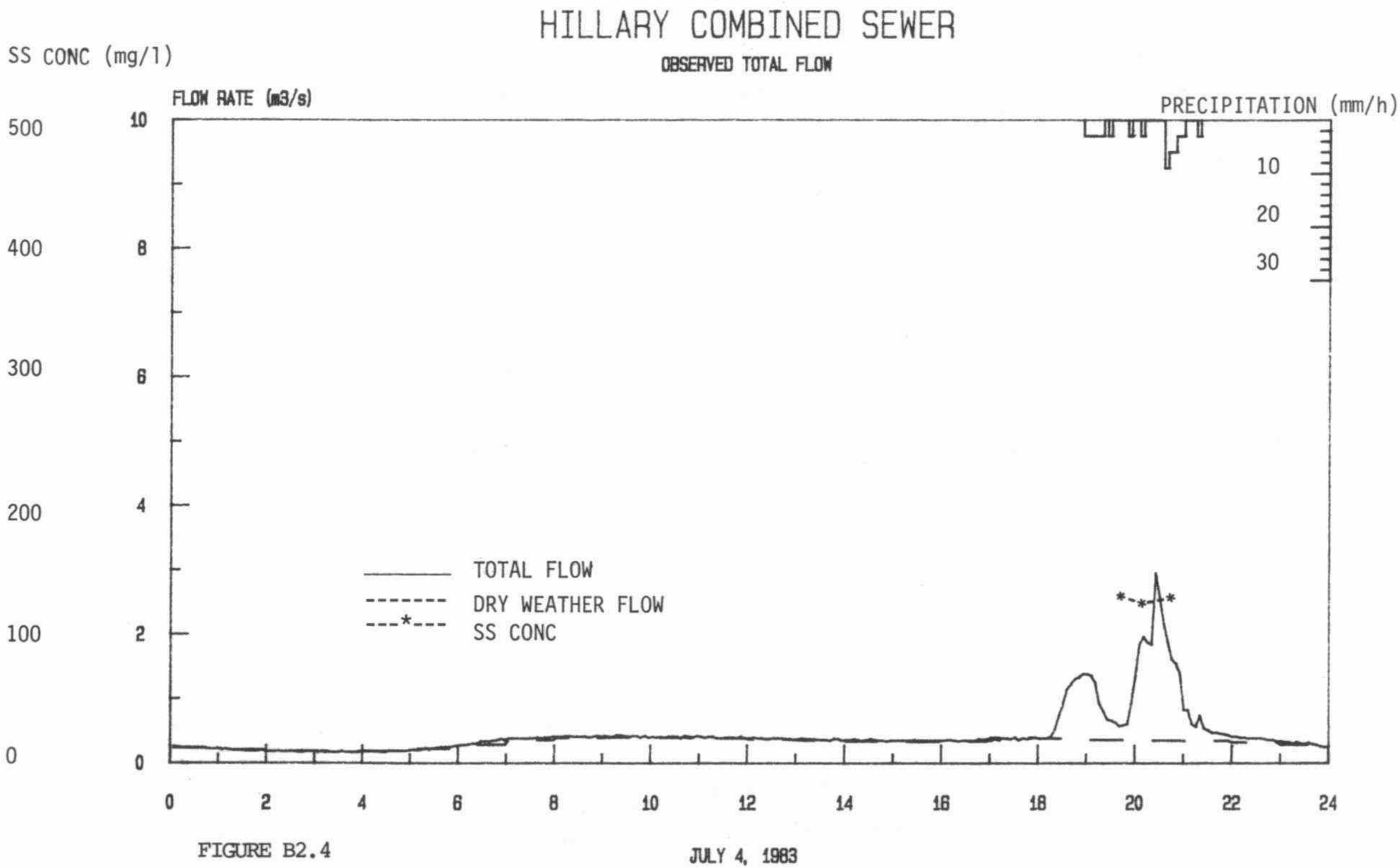

HILLARY COMBINED SEWER
OBSERVED TOTAL FLOW
SS CONC (mg/l)
FLOW RATE (m3/s)
PRECIPITATION (mm/h)
500
400
300
200
100
0
10
8
6
4
2
0
10
20
30
TOTAL FLOW
DRY WEATHER FLOW
SS CONC
0
2
4
6
8
10
12
14
16
18
20
22
24
JULY 4, 1983

FIGURE B2.4

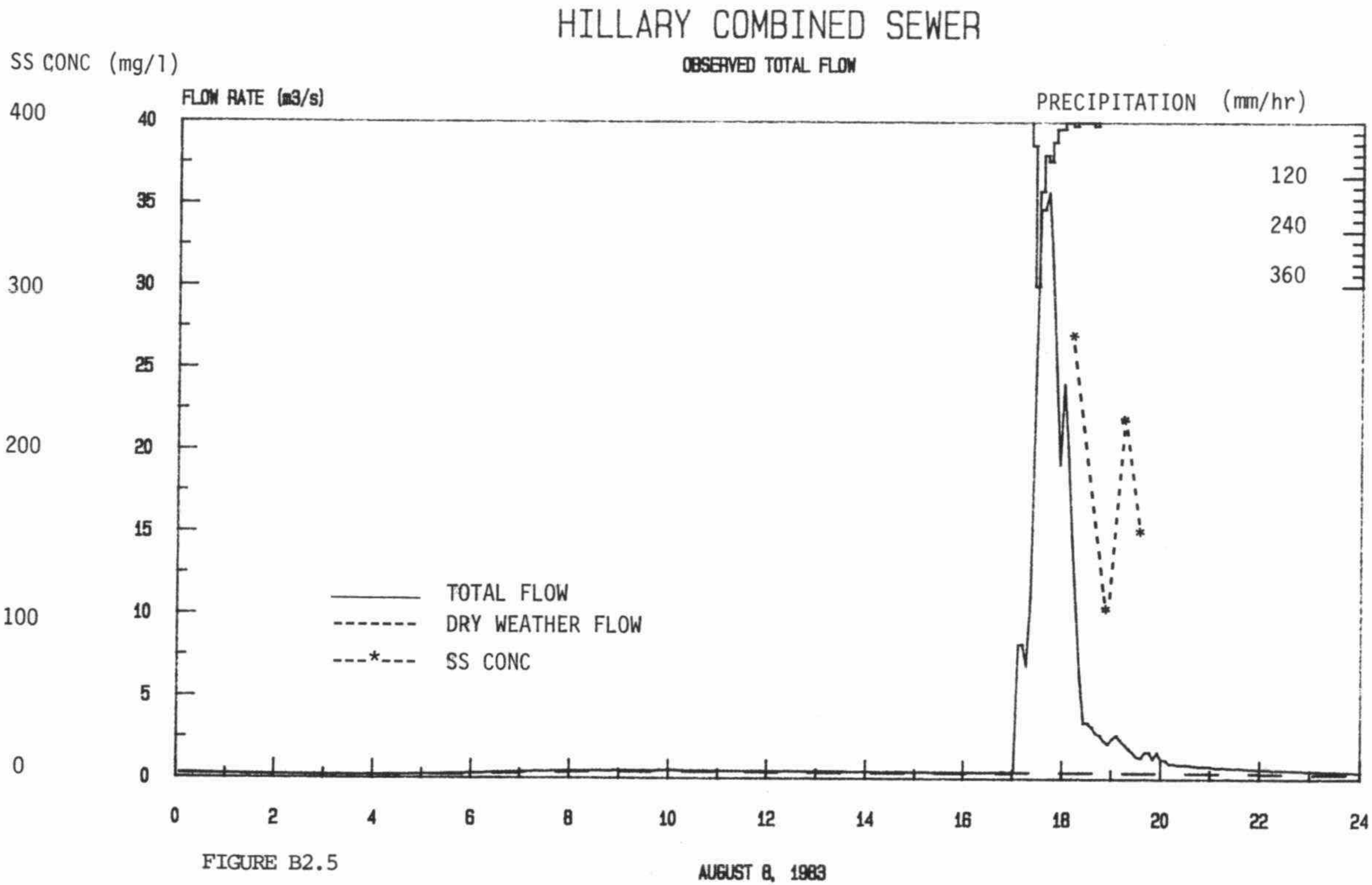

HILLARY COMBINED SEWER
OBSERVED TOTAL FLOW
SS CONC (mg/l)
FLOW RATE (m3/s)
PRECIPITATION (mm/hr)
400
300
200
100
0
40
35
30
25
20
15
10
5
0
120
240
360
TOTAL FLOW
DRY WEATHER FLOW
---*--- SS CONC
0
2
4
6
8
10
12
14
16
18
20
22
24
AUGUST 8, 1983

FIGURE B2.5

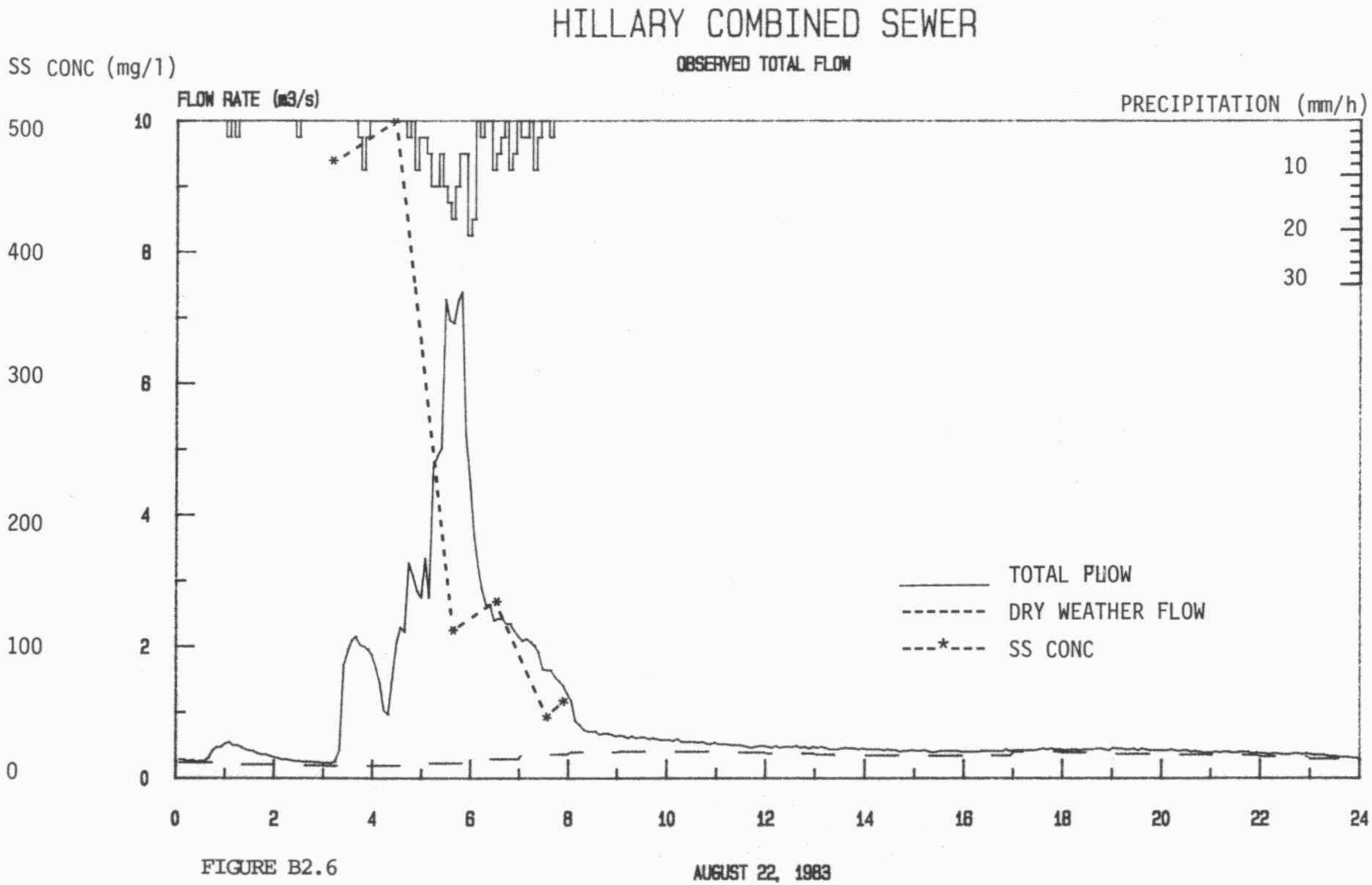
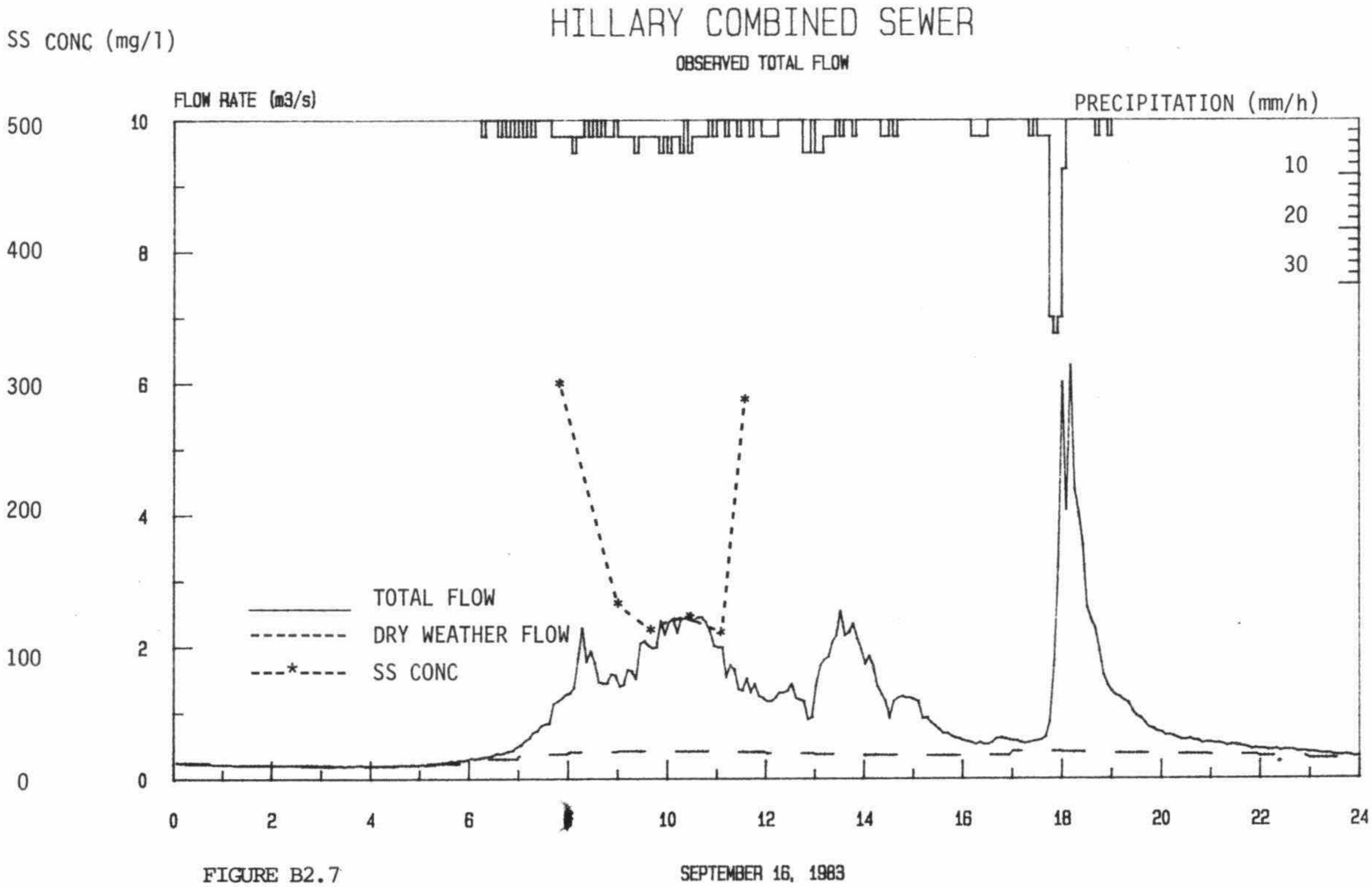
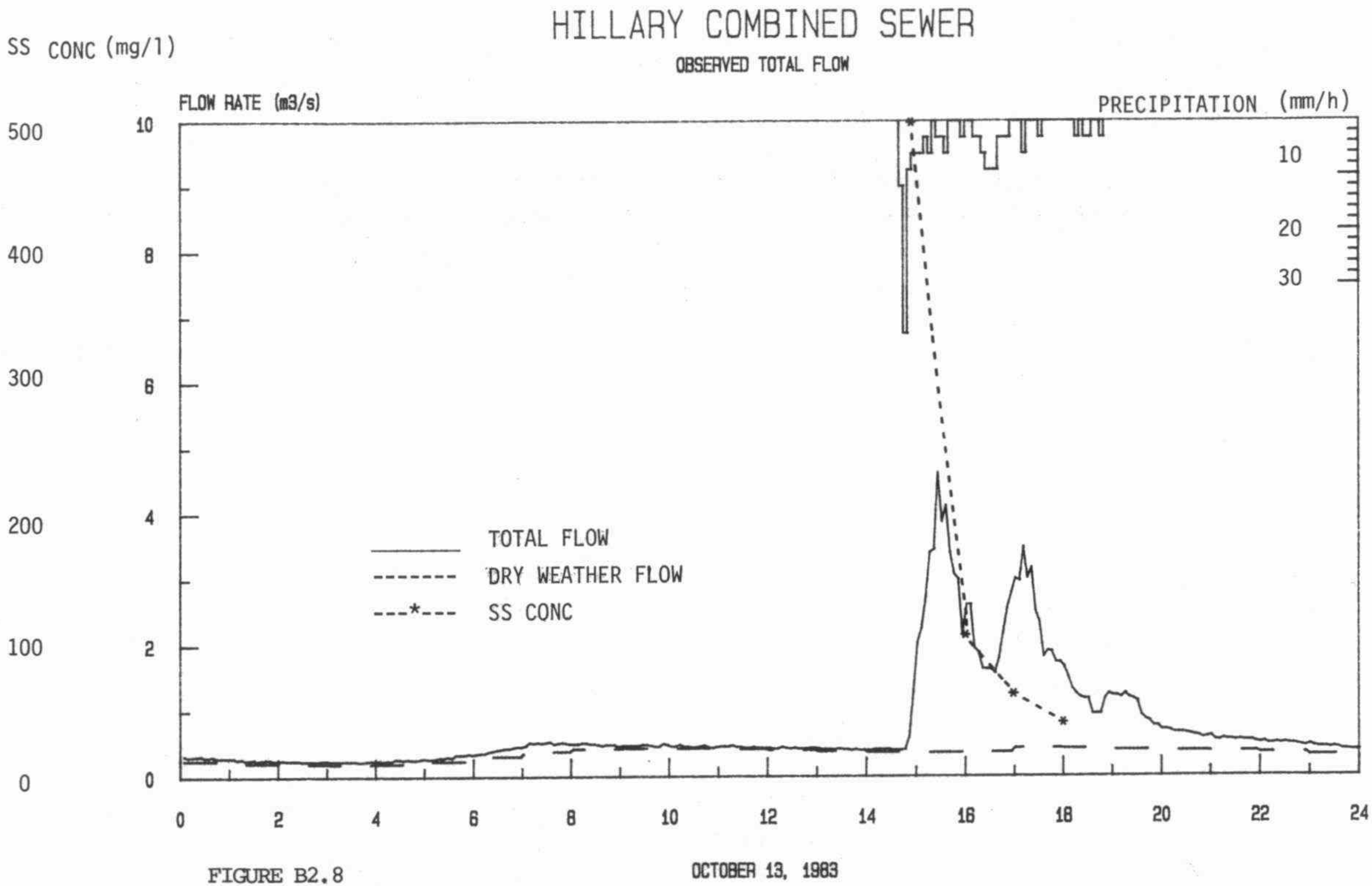

HILLARY COMBINED SEWER
OBSERVED TOTAL FLOW
SS CONC (mg/l)
FLOW RATE (m3/s)
PRECIPITATION (mm/h)
500
400
300
200
100
0
10
8
6
4
2
0
10
20
30
TOTAL FLOW
DRY WEATHER FLOW
---*--- SS CONC
0
2
4
6
8
10
12
14
16
18
20
22
24
AUGUST 22, 1983

FIGURE B2.6



HILLARY COMBINED SEWER
OBSERVED TOTAL FLOW
SS CONC (mg/l)
FLOW RATE (m3/s)
PRECIPITATION (mm/h)
500
400
300
200
100
0
10
8
6
4
2
0
10
20
30
TOTAL FLOW
DRY WEATHER FLOW
---*---- SS CONC
0
2
4
6
8
10
12
14
16
18
20
22
24
SEPTEMBER 16, 1983

FIGURE B2.7



HILLARY COMBINED SEWER
OBSERVED TOTAL FLOW
SS CONC (mg/l)
FLOW RATE (m3/s)
PRECIPITATION (mm/h)
500
400
300
200
100
0
10
8
6
4
2
0
10
20
30
TOTAL FLOW
DRY WEATHER FLOW
---*--- SS CONC
0
2
4
6
8
10
12
14
16
18
20
22
24
OCTOBER 13, 1983

FIGURE B2.8

TABLE B2.4

OBSERVED COMBINED SEWAGE DATA (Sheet 1 of 5)

Sampling Station: 3 Event Date: 830519

Sample Time	Flow Rate m3/s	PPUT mg/l	PPO4FR mg/l	RSP mg/l	BOD5 mg/l	Cd ug/l	Cu ug/l	Pb ug/l	Zn ug/l
14:15	1.928	3.45	0.80	218.0	94.0	10	160	280	420
14:45	4.225	2.38	0.40	269.0	62.0	10	180	290	390
15:15	4.305	1.60	0.16	263.0	37.0	10	100	160	250
15:45	2.715	1.10	0.14	164.0	24.8	10	70	90	180
16:15	2.258	0.72	0.10	76.2	15.9	10	50	60	150
16:45	2.785	0.56	0.08	70.0	13.8	10	50	60	130
17:25	1.932	0.54	0.06	47.3	16.3	10	60	60	110
18:05	2.142	0.62	0.06	46.7	19.0	10	60	60	120

Sampling Station: 3 Event Date: 830529

Sample Time	Flow Rate m3/s	PPUT mg/l	PPO4FR mg/l	RSP mg/l	BOD5 mg/l	Cd ug/l	Cu ug/l	Pb ug/l	Zn ug/l
14:40	1.737	5.10	2.26	317.0	267.0				
15:10	4.694	2.72	0.50	343.0	82.2				
15:40	4.041	1.70	0.30	216.0	63.2				
16:10	1.278	1.48	0.34	134.0	45.4				
16:40	1.992	1.35	0.38	92.2	36.8				

Notes :
PPUT = Total Phosphorous
PPO4FR = Filtered Reactive Phosphorous
RSP = Residue Particulate.

TABLE B2.4

OBSERVED COMBINED SEWAGE DATA (Sheet 2 of 5)

Samlplng Station: 3　　　　Event Date: 830606

Sample Time	Flow Rate m3/s	PPUT mg/l	PPO4FR mg/l	RSP mg/l	BOD5 mg/l	Cd ug/l	Cu ug/l	Pb ug/l	Zn ug/l
09:15	1.199	3.75	1.64	214	103.0				
10:00	1.897	2.75	1.08	201	70.3				
10:45	1.868	1.65	0.50	132	38.6				
11:30	1.928	1.33	0.38	114	21.6				
12:00	1.694	1.38	0.36	104	21.4				

Sampling Station: 3　　　　Event Date: 830704

Sample Time	Flow Rate m3/s	PPUT mg/l	PPO4FR mg/l	RSP mg/l	BOD5 mg/l	Cd ug/l	Cu ug/l	Pb ug/l	Zn ug/l
20:00	1.331	1.70	0.28	135.0	51.0	5	120	170	300
20:15	1.868	1.75	0.30	126.0	58.0	5	90	170	260
20:30	2.024	1.85	0.30	135.0	55.0	5	120	180	280
20:45	1.637	1.85	0.28	131.0	58.0	5	340	180	340

Notes :
PPUT = Total Phosphorous　　　　RSP = Residue Particulate.
PPO4FR = Filtered Reactive Phosphorous

TABLE B2.4

OBSERVED COMBINED SEWAGE DATA (Sheet 3 of 5)

Samlplng Station: 3 Event Date: 830808

Sample Time	Flow Rate m3/s	PPUT mg/l	PPO4FR mg/l	RSP mg/l	BOD5 mg/l	Cd ug/l	Cu ug/l	Pb ug/l	Zn ug/l
17:20	2.3563	3.70	1.60	1120.0	145.0	5	150	70	160
17:50	6.3143	2.75	0.10	429.0	45.6	5	160	640	910
18:20	7.012	2.30	0.06	271.0	43.2	5	130	500	710
18:50	2.116	1.75	0.06	103.0	34.8	5	130	410	570
19:20	1.855	2.13	0.04	214.0	41.5	5	130	430	820
19:50	1.224	1.30	0.02	150.0	42.1	5	110	380	610

Sampling Station: 3 Event Date: 830811

Sample Time	Flow Rate m3/s	PPUT mg/l	PPO4FR mg/l	RSP mg/l	BOD5 mg/l	Cd ug/l	Cu ug/l	Pb ug/l	Zn ug/l
06:00	NA	2.50	0.98	135.0		4	120	110	230
06:15	NA	2.00	0.68	179.0		4	100	150	240
06:30	NA	2.05	0.68	135.0		4	140	140	200
06:45	NA	2.05	0.52	133.0		4	110	110	170
07:00	NA	2.00	0.56	84.1		4	110	90	170
07:15	NA	2.03	0.58	76.1		5	70	130	170

Notes :
PPUT = Total Phosphorous
RSP = Residue Particulate.
PPO4FR = Filtered Reactive Phosphorous
NA = Not Available, Flow Recorder Malfunction.

TABLE B2.4

OBSERVED COMBINED SEWAGE DATA (Sheet 4 of 5)

Samlplng Station: 3 Event Date: 830822

Sample Time	Flow Rate m3/s	PPUT mg/l	PPO4FR mg/l	RSP mg/l	BOD5 mg/l	Cd ug/l	Cu ug/l	Pb ug/l	Zn ug/l
03:25	2.598	3.2	0.56	465.0	148.0	4	310	530	720
04:25	2.872	5.03	0.80	609.0	173.0	5	180	420	740
05:25	3.990		0.12	110.0	25.8	4	70	110	180
06:25	2.167		0.10	124.0	13.4	4	50	70	150
07:25	1.642		0.18	44.1	13.6	4	40	50	160
07:50	1.403		0.44	59.2	18.6	4	50	40	140

Sampling Station: 3 Event Date: 830916

Sample Time	Flow Rate m3/s	PPUT mg/l	PPO4FR mg/l	RSP mg/l	BOD5 mg/l	Cd ug/l	Cu ug/l	Pb ug/l	Zn ug/l
08:10	1.674	4.46	1.24	294.0	111.0	6	240	220	450
08:55	1.606	2.02	0.56	126.0	41.0	6	170	130	290
09:40	1.830	1.52	0.46	106.0	40.0	6	150	80	240
10:30	1.857	1.30	0.36	114.0	38.0	6	160	160	270
11:10	1.758	2.50	0.32	106.0	32.0	6	120	90	220
11:50	1.476	4.75	1.40	288.0	120.0	6	350	220	470

Notes :
PPUT = Total Phosphorous
RSP = Residue Particulate
PPO4FR = Filtered Reactive Phosphorous

TABLE B2.4

OBSERVED COMBINED SEWAGE DATA (Sheet 5 of 5)

Samlplng Station: 3 Event Date: 831012

Sample Time	Flow Rate m3/s	PPUT mg/l	PPO4FR mg/l	RSP mg/l	BOD5 mg/l	Cd ug/l	Cu ug/l	Pb ug/l	Zn ug/l
05:15	1.760	4.03	1.04		142.0	6	160	120	260
05:45	1.785	4.10	1.06		150.0	6	180	160	280
06:15	1.674	1.55	0.32		72.0	6	130	150	200
06:45	1.642	2.15	0.22		38.0	6	110	160	210
08:40	1.714	1.00	0.22		28.0	5	60	90	110
09:00	1.356	0.65	0.18		27.0	5	70	90	130

Sampling Station: 3 Event Date: 831013

Sample Time	Flow Rate m3/s	PPUT mg/l	PPO4FR mg/l	RSP mg/l	BOD5 mg/l	Cd ug/l	Cu ug/l	Pb ug/l	Zn ug/l
15:00	2.901	3.40	0.54	525.0	95.0	5	160	230	390
16:00	2.506	0.84	0.16	105.0	54.0	5	60	130	180
17:00	2.901	0.89	0.38	71.6	23.0	5	50	100	140
18:00	1.447	0.80	0.26	50.3	48.0	5	50	80	120
						5			
						5			

Notes :
PPUT = Total Phosphorous RSP = Residue Particulate
PPO4FR = Filtered Reactive Phosphorous

TABLE B2.5

BACTERIAL CONCENTRATIONS IN COMBINED SEWAGE (Sheet 1 of 2)

Event	Site	Date	F.C. Counts,100ml.	Log	F.S. Counts,10ml.	Log
6452	8	830830	980000	5.591	870000	5.940
	8		9400000	6.973	730000	5.863
	8		9600000	6.982	760000	5.881
	8		530000	5.724	420000	5.623
	8		1000	3.000	230000	5.362
	9		470000	6.672	190000	5.279
	9		480000	6.681	75000	4.875
	9		460000	6.663	210000	5.322
	3		830000	6.919	910000	5.959
	8		530000	6.724	1900000	6.279
	9		3100000	6.491	390000	5.591
5377	3	830808	9400000	6.973	75000	4.875
	3		670000	5.826	31000	4.491
	3		690000	5.839	41000	4.613
	3		430000	5.633	39000	4.591
	3		500000	5.699	35000	4.544
	3		450000	5.653	37000	4.568
5375	3	830804	3300000	6.519	380000	5.580
	3		2300000	6.362	380000	5.580
	3		2000000	6.301	290000	5.462
	3		3000000	6.477	350000	5.544
5370	3	830704	3400000	6.531	240000	5.380
	3		3900000	6.591	360000	5.556
	3		3900000	6.591	370000	5.568
	3		4400000	6.643	310000	5.491
	8		1.33E+07	7.124	1180000	6.072
	8		1.43E+07	7.155	650000	5.813
	8		1.36E+07	7.134	680000	5.833
	8		8900000	6.949	1080000	6.033
	8		9600000	6.982	1430000	6.155
	8		1.26E+07	7.100	1490000	6.173
	9		8400000	6.924	1250000	6.097
	9		8300000	6.919	180000	5.255
	9		9200000	6.964	240000	5.380
	9		5900000	6.771	260000	5.415
5367	3	830606	8700000	5.940	130000	5.114
	8		4700000	5.672	150000	5.176
	9		9700000	5.987	170000	5.230
5366	3	830529	8300000	6.919	770000	5.886
			1900000	6.279	420000	5.623
			1060000	6.025	250000	5.398
			9100000	5.959	150000	5.176
			1600000	6.204	113000	5.053
			1270000	6.104	83000	4.919

TABLE B2.5

BACTERIAL CONCENTRATIONS IN COMBINED SEWAGE (Sheet 2 of 2)

Event	Site	Date	F.C. Counts,100ml.	Log	F.S. Counts,10ml.	Log
5394	3	830811	370000	5.568	280000	5.447
	3		190000	5.279	72000	4.857
	3		180000	5.255	95000	4.978
	3		96000	4.982	52000	4.716
	3		460000	5.663	64000	4.806
	3		480000	5.681	47000	4.672
	3		97000	4.987	94000	4.973
	8		240000	5.380	2200000	6.342
	8		22000	4.342	870000	3.940
	9		32000	4.505	72000	4.857
	9		360000	5.556	46000	4.663
5393	3	830811	5300000	6.724	650000	5.813
	3		5500000	6.740	460000	5.663
	3		4200000	6.623	280000	5.447
	3		3300000	6.519	290000	5.462
	3		2900000	6.462	270000	5.431
	3		2700000	6.431	230000	5.362
	8		7100000	6.851	360000	5.556
	8		4300000	6.633	320000	5.505
	8		3900000	6.591	220000	5.342
	8		2800000	6.447	290000	5.462
	8		2900000	6.462	310000	5.491
6467	3	831115	1050000	6.021	220000	5.342
			850000	5.929	140000	5.146
			710000	5.851	110000	5.041
6466	3	831116	1700000	6.230	100000	5.000
6463	3	831115	990000	5.996	5500000	6.740
6457	3	831013	3400000	6.531	360000	5.556
	3		1080000	6.033	150000	5.176
	3		1240000	6.093	320000	5.505
	3		2300000	6.362	530000	5.724
6455	3	381012	6100000	6.785	730000	5.868
	3		5700000	6.756	940000	5.973
	3		1190000	6.076	300000	5.477
	3		930000	5.968	270000	5.431
	3		580000	5.763	220000	5.342
	3		970000	5.987	230000	5.362
Log Mean				6.218		5.421
Std. Dev.				0.700		0.462

APPENDIX B3

INFLOW / INFILTRATION

WET WEATHER I/I FROM SANITARY SEWER AREA

Method of Calculation

(1) Daily volumes of dry days were extracted from WPCP records

  (a) there had to be at least 2 days pass after a precipitation event.

(2) The average daily dry weather flow for individual months and each year were calculated.

(3) The wet weather days were identified

  (a) any day with precipitation
  (b) plus the day after the precipitation.

(4) The net extraneous flow was calculated

  (a) the average monthly dry weather flow was subtracted from the individual wet flow record for each day between April and October.

(5) The wet days were selected for the regression table

  (a) precipitation had to be greater than 5.0 mm

  (b) the net extraneous flow for the day of precipitation had to have a positive value

  (c) the net extraneous flow for the day after the precipitation event had to have a positive value or, if negative, the sum of the two days had to be positive

  (d) if two consecutive events greater than 5.0 mm occurred then

    1) the two days of precipitation were added together

2) the two days of flow plus the day after were added together.

(6) Use equation of the form:

$$II = A * P^B,$$

where II = Event I/I in 1,000 m^3

P = Event precipitation in mm

A and B are constants

(7) Obtain regression results for the April-October season for each year from 1980 through 1983. Select the year with best coefficient of correlation.

(8) Apportion II volume to each hour of event according to precipitation distribution. Synthesize hydrograph for the distributed II, using a unit hydrograph of the WPCP.

TABLE B3.1

WET WEATHER I/I AT HUMBER WPCP (1)

Month (1980)	Avg DWF for month ($10^3 m^3$)	Event I/I Volume ($10^3 m^3$)	Precipitation (mm)
April	350.9	89.2	15.2
		434.3	29.4
		242.4	20.1
May	333.6	61.3	12.8
		39.3	20.5
June	408.8	42.6	20.5
July	359.0	124.8	7.3
		341.7	25.2
		672.9	62.1
August	391.1	172.0	5.8
September	427.1	16.3	9.1
		90.0	6.2
		85.9	14.4
		1.7	5.1
October	456.4	51.2	18.8
		2.9	5.0
		124.1	35.4

$I/I = 1.475 * P^{1.446}$

Where I/I = Event inflow/infiltration, $10^3 m^3$

P = Event precip. mm.

Coeff. of correl. = 0.663

Note:

(1) For events greater than 5.0 mm.

APPENDIX C1

MODEL INPUT

TABLE C 1.1 (Sheet 1 of 6)

```
  10   8
   1   2   3   4
RUNOFF
A1   *** TAWMS COMBINED SEWER ANALYSIS *** HUMBER SEWERSHED
A1   1983 CATCHMENTS
B1   0 1  0     1     0     1     1    16     0    11    07    79
B2        40  15.   90. 0.01
E1         4 60.0
E2       7.8 28.1    .0    .2
F1       0.0  0.0  1.3  3.3  3.3  4.1  7.0  4.1  2.2  0.8  0.1  0.0
G1       331  337    3
G1       332  330    3
G1       333  330    3
G1       334  330    3
G1       335  330    3
G1       336  337    3
G1       337  330    3
G1       341  347    3
G1       342  340    3
G1       343  340    3
G1       344  340    3
G1       345  340    3
G1       346  347    3
G1       347  340    3
G1       351  357    3
G1       352  350    3
G1       353  350    3
G1       354  350    3
G1       355  350    3
G1       356  357    3
G1       357  350    3
G1       401  410    3
G1       402  410    3
G1       403  410    3
G1       404  410    3
G1       405  410    3
G1       501  510    3
G1       504  510    3
G1       505  510    3
G2
H1        -1         1.0
H1        -2                          .0150  .30 -2.0 10.0  7.3 0.50 .00025
H1   1 2531   3311351.135.1 18.9   .06
H1   1 2532   332    9.   0.9 50.0   .06
H1   1 2533   333 184. 18.4100.0   .06
H1   1 2534   334 217. 21.7 42.0   .06
H1   1 2535   335 222. 22.2  5.0   .06
H1   1 2536   336 321. 32.1 18.9   .06                7.00
H1   1 2541   3413018.301.8 18.9   .06
H1   1 2542   342  20.  2.0 50.0   .06
H1   1 2543   343 411. 41.1100.0   .06
H1   1 2544   344 485. 48.5 42.0   .06
H1   1 2545   345 496. 49.6  5.0   .06
H1   1 2546   346 717. 71.7 18.9   .06                7.00
H1   1 2551   351 808. 80.8 18.9   .06
H1   1 2552   352    6.    .6 50.0   .06
H1   1 2553   353 110. 11.0100.0   .06
H1   1 2554   354 130. 13.0 42.0   .06
H1   1 2555   355 133. 13.3  5.0   .06
```

```
H1  1 2556  356 192. 19.2 18.9  .06                  7.00
H1  1 2151  4011489.101.5 33.0  .02
H1  1 2152  402  59.  4.0 50.0  .02
H1  1 2153  403 213. 14.5 100.  .02
H1  1 2154  404 499. 34.0 90.0  .02
H1  1 2155  405 191. 13.0 10.0  .02
H1  1 2181  5011452.149.8 28.0 .015
H1  1 2184  504 237. 24.5 95.0 .015
H1  1 2185  505 211. 21.8  2.0 .015
H2
J1  8    5    0    0  13.  0.0            0.30
J2           -2           200.       0.40        26.7  7.0         7.0
J2  FAMILY    0    0                                        .189
J2 MFAMILY    0    0                                        .500
J2COMMERCE    0    0                                        1.00
J2INDUSTRY    0    0                                        .420
J2  OTHERS    0    0                                        .050
J3                  -2                                                          .3
J3 S.S.      MG/L   0  0 2     1000.1000.1000.1000.1000.1.6622.138
J3 BOD5      MG/L   0  0 2      200. 200. 200. 200. 200.1.5001.164
J3 SOLU-P.   MG/L   0  4 1                                 1.0 .432
J3 TOTAL-P   MG/L   0  0 2      9.2  9.2  9.2  9.2  9.21.662.0197
J3 CADMIUM   MG/L   0  0 2     .038 .038 .038 .038 .0381.6628.E-5
J3 COPPER    MG/L   0  0 2     .645 .645 .645 .645 .6451.6621.E-3
J3 LEAD      MG/L   0  0 2     1.13 1.13 1.13 1.13 1.131.6622.E-3
J3 ZINC      MG/L   0  0 2     1.89 1.89 1.89 1.89 1.891.6624.E-3
J5
L1      2531    1          19.8
L1      2532    2           0.5
L1      2533    3          14.6
L1      2534    4           7.3
L1      2535    5           1.0
L1      2536    1           4.7
L1      2541    1          44.3
L1      2542    2           1.2
L1      2543    3          32.6
L1      2544    4          16.3
L1      2545    5           2.3
L1      2546    1          10.5
L1      2551    1          11.8
L1      2552    2           0.3
L1      2553    3           8.7
L1      2554    4           4.4
L1      2555    5           0.6
L1      2556    1           2.8
L1      2151    1          14.2
L1      2152    2           0.8
L1      2153    3           6.2
L1      2154    4          13.0
L1      2155    5           0.6
L1      2181    1          20.6
L1      2184    4          11.4
L1      2185    5           0.2
M1  5    1
M2330  340  350  410  510
ENDPROGRAM
```

TABLE C 1.1 (Sheet 3 of 6)

```
   8   9
   1   2
TRANSPORT
A1
C1   *** TAWMS.COMBINED SEWER ANALYSIS *** HUMBER SEWERSHED
C1      1983 REGULATORS.
D1        40     4     0   13     0     1    4     4     790711     1     1
D2      900.        .001         13.       16.00        .01
D3  0     0     0     0
E1 330                  16 0.08140    0.014    0.160     0.06     0.22
E1 335 330              18              .42                                          336.0
E1 336 335              16
E1 340                  16  0.1822    0.014    0.160     0.06     0.22
E1 345 336 340          16
E1 350                  16 0.04860    0.014    0.160     0.06     0.22
E1 355 350              18             0.27                                          356.0
E1 356 355              16
E1 360 345              16
E1 362 360              18             2.75                                          363.0
E1 363 362 356          16
E1 364 363              18             1.64                                          365.0
E1 365 364              16
E1 410                  16 0.06300    0.014    0.160     0.06     0.22
E1 420 410              18            0.324                                          422.0
E1 422 420              16
E1 510                  16 0.05000    0.014    0.160     0.06     0.22
E1 520 510              18            0.474                                          522.0
E1 522 520              16
E1  50                  16   .6883    0.014    0.370     0.04     0.17
E1 850  50               1   1720.    1.634    .1663    .0209              1.0
E1  42 850              16   .3846    0.014    0.370     0.04     0.17
E1 842  42               1   3950.    1.727    .2846    .0209              1.0
E1  24 842              16
E1 824  24               1    930.    1.524    .5054    .0209              1.0
E1  21 824              16    .368    0.014    0.370     0.04     0.17
E1 821  21               1   1750.    1.667    .5600    .0209              1.0
E1  14 821              16    .104    0.014    0.370     0.04     0.17
E1 814  14               1    870.    1.829    .2529    .0209              1.0
E1  11 814 860          16
E1  10  11 522          16   .0129    0.014    0.370     0.04     0.17
E1 810  10               1   2310.    2.800    .2251    .0209              1.0
E1   6 810              16    .525    0.014    0.370     0.04     0.17
E1 806   6               1   1980.    3.500    .1414    .0209              1.0
E1   2 806              16    .116    0.014    0.370     0.04     0.17
E1 770   2              20   10.15     2.27     38.1    26.22    3.189                  1.
E1 802   1               1    350.    3.500    .2097    .0209              1.0
E1   1 770              16
E1  94                  16    .644    0.014    0.370     0.04     0.17
E1  96                  16   .0388    0.014    0.370     0.04     0.17
E1  98 802  94  96      16
E1 772  98              16
E1 774 772              18            11.73                                          776.
E1 776 774              16
E1 150                  16    1.00    0.014    0.370     0.04     0.17
E1 155 365 150 220      16
E1 220                  16   .0171
E1 160 155 422          16
E1 860 160              16
E1  99 776              16
```

TABLE C 1.1 (Sheet 4 of 6)

```
E2
F1  5CADMIUM   MG/L     0
F1  6COPPER    MG/L     0
F1  7LEAD      MG/L     0
F1  8ZINC      MG/L     0
I1 24  160    6  772
J2330  340  350  363  365  410  422  510  522    2    1  772  776
R1     16.0       0.0
R1     16.0       0.0
R1     16.0       0.0
R1     16.0       0.0
R1     20.5       4.44
R1     20.5       3.79
R1     20.5       4.61
R1     20.5       3.62
R1     26.0       0.0
R1     26.0       0.0
R1     26.0       0.0
R1     26.0       0.0
ENDPROGRAM
```

TABLE C 1.1 (Sheet 5 Of 6)

```
   8   9
   1   2
TRANSPORT
A1
C1  *** TAWMS.COMBINED SEWER ANALYSIS *** HUMBER SEWERSHED
C1      1983 REGULATORS.
D1       40    4    0   13    0    1    4    4    790711    1    1
D2     900.       .001       13.      16.00        .01
D3  0    0    0    0
E1 330               16 0.08140   245.4   208.9    2.42    5.42
E1 335 330           18              .42                                   336.0
E1 336 335           16
E1 340               16  0.1822   245.4   208.9    2.42    5.42
E1 345 336 340       16
E1 350               16 0.04860   245.4   208.9    2.42    5.42
E1 355 350           18             0.27                                   356.0
E1 356 355           16
E1 360 345           16
E1 362 360           18             2.75                                   363.0
E1 363 362 356       16
E1 364 363           18             1.64                                   365.0
E1 365 364           16
E1 410               16 0.06300   245.4   208.9    2.42    5.42
E1 420 410           18            0.324                                   422.0
E1 422 420           16
E1 510               16 0.05000   245.4   208.9    2.42    5.42
E1 520 510           18            0.474                                   522.0
E1 522 520           16
E1  50               16   .6883   173.8   191.0    2.58    5.66
E1 850  50            1   1720.   1.634   .1663   .0209           1.0
E1  42 850           16   .3846   173.8   191.9    2.58    5.66
E1 842  42            1   3950.   1.727   .2846   .0209           1.0
E1  24 842           16
E1 824  24            1    930.   1.524   .5054   .0209           1.0
E1  21 824           16    .368   173.8   191.9    2.58    5.66
E1 821  21            1   1750.   1.667   .5600   .0209           1.0
E1  14 821           16    .104   173.8   191.9    2.58    5.66
E1 814  14            1    870.   1.829   .2529   .0209           1.0
E1  11 814 860       16
E1  10  11 522       16   .0129   173.8   191.9    2.58    5.66
E1 810  10            1   2310.   2.800   .2251   .0209           1.0
E1   6 810           16    .525   173.8   191.9    2.58    5.66
E1 806   6            1   1980.   3.500   .1414   .0209           1.0
E1   2 806           16    .116   173.8   191.9    2.58    5.66
E1 770   2           20   10.15    2.27    38.1   26.22   3.189                1.
E1 802   1            1    350.   3.500   .2097   .0209           1.0
E1   1 770           16
E1  94               16    .644   173.8   191.9    2.58    5.66
E1  96               16   .0388   173.8   191.9    2.58    5.66
E1  98 802  94  96   16
E1 772  98           16
E1 774 772           18            11.73                                   776.
E1 776 774           16
E1 150               16    1.00   173.8   191.9    2.58    5.66
E1 155 365 150 220   16
E1 220               16   .0171   1067.   2077.    19.4
E1 160 155 422       16
E1 860 160           16
E1  99 776           16
```

TABLE C 1.1 (Sheet 6 of 6)

```
E2
F1  1S.S.      MG/L     0
F1  2BOD5      MG/L     0
F1  3SOL.P     MG/L     0
F1  4TOTAL-P   MG/L     0
I1 24  160    6  772
J2330  340  350  363  365  410  422  510  522    2    1  772  776
R1     16.0      0.0
R1     16.0      0.0
R1     16.0      0.0
R1     16.0      0.0
R1     20.5      4.44
R1     20.5      3.79
R1     20.5      4.61
R1     20.5      3.62
R1     26.0      0.0
R1     26.0      0.0
R1     26.0      0.0
R1     26.0      0.0
ENDPROGRAM
```

APPENDIX C2

DERIVATION OF RUNOFF DATA

TABLE C 2.1

FLOW RATES AND SS LOAD RATES OF RUNOFF

Date & Time	Flow Rate Q	SS Load Rate	Date & Time	Flow Rate Q	SS Load Rate
830519			830529		
14:15	1.616	343764	14:40	1.425	474089
14:45	3.913	1059980	15:10	4.382	1533500
15:15	3.993	1055670	15:40	3.729	796316
15:45	2.403	368720	16:10	0.966	94712
16:15	1.946	95519	16:40	1.680	107122
16:45	2.473	118410			
17:25	1.620	14843			
18:05	1.883	25966			
830606			830704		
09:15	0.887	180046	20:00	1.019	103145
10:00	1.585	304757	20:15	1.556	158828
10:45	1.556	170036	20:30	1.712	196700
11:30	1.687	151346	20:45	1.325	137907
12:00	1.453	107020			
830808			830822		
18:20	6.700	1823710	03:25	2.286	1131530
18:50	1.804	141408	04:25	2.560	1672510
19:20	1.543	320430	05:25	3.678	362360
19:50	0.912	107069	06:25	1.855	192168
			07:50	1.091	6517
830916			831013		
08:10	1.362	415616	15:00	2.589	1446480
08:55	1.294	125816	16:00	2.194	186590
09:40	1.518	117440	17:00	2.589	131171
10:30	1.545	135158			
11:10	1.446	109808			
11:50	1.164	348548			

Loadrate = 77379 * Q^1.66
Where Loadrate is in mg/s; Q is in m^3/s
Coefficient of Correlation = 0.62
Notes: (1) Derived from station 1 and station 3 data.
(2) Runoff = Combined sewage less sanitary sewage.

TABLE C 2.2

FLOW RATES AND BOD5 LOAD RATES OF RUNOFF (1)

Date & Time	Flow Rate Q	BOD5 Load Rate	Date & Time	Flow Rate Q	BOD5 Load Rate
830519			830529		
14:15	1.616	116076	15:10	4.38	320695
14:45	3.913	196794	15:40	3.72	190235
15:15	3.993	94129	16:40	1.68	8150
15:45	2.403	2176			
830606			830704		
09:15	0.887	58341	20:00	1.01	2725
10:00	1.585	68203	20:15	1.55	43188
10:45	1.556	6949	20:30	1.71	46164
			20:45	1.32	29790
830808			830822		
18:20	6.700	237763	03:25	2.28	319348
18:50	1.804	8481	04:25	2.56	431700
19:20	1.543	11827	05:25	3.67	37786
830916			831012		
08:10	1.362	120658	05:15	1.44	184764
08:55	1.294	690	05:45	1.47	202594
09:40	1.518	8044	06:15	1.36	55372
10:30	1.545	5410			
11:50	1.164	111964			
831013					
15:00	2.589	210439			
16:00	2.194	70168			
17:00	2.589	1567			
18:00	1.135	4300			

```
*******************************
*  Loadrate = 13736 * Q^1.50  *
*  Where Loadrate is in mg/s  *
*  Q is in m3/s               *
*  Coefficient of             *
*  Correlation = 0.40         *
*  Notes:(1) Derived from st.1*
*  and 3 data.(2) Runoff=Comb.*
*  sewage less sanitary sewage*
*******************************
```

TABLE C2.3

POLLUTANT RATIOS
TOTAL PHOSPHOROUS vs SUSPENDED SOLIDS IN RUNOFF (1)

Date & Time	TP (ug/l)	SS (mg/l)	Date & Time	TP (ug/l)	SS (mg/l)
830519			830529		
14:15	3.1	212.7	14:40	5.0	332.7
14:45	2.1	270.9	15:10	2.5	349.9
15:15	1.3	264.4	15:40	1.4	213.5
15:45	0.5	153.4	16:10	0.2	98.0
16:15			16:40	0.6	63.8
16:45					
17:25					
18:05					
830606			830704		
0915	3.2	203.0	20:00	0.6	101.2
1000	2.2	192.3	20:15	1.0	102.1
1045	0.9	109.3	20:30	1.2	114.9
1130	0.6	89.7	20:45	1.0	104.1
1200	0.5	73.6			
830808			830822		
18:20	2.2	272.2	03:25	2.9	495.0
18:50	1.1	78.4	04:25	5.0	653.3
19:20	1.5	207.7			
19:50					
830916			831013		
08:10	4.2	305.1	15:00	3.2	558.7
08:55	1.2	97.1	16:00	0.2	85.0
09:40	0.7	77.4	17:00	0.3	50.7
10:30	0.5	87.5	18:00		
11:10	1.9	75.9			
11:50	4.6	299.4			

Mean Ratio = 9.172 ug/mg
Std. Dev. = 4.704 ug/mg

Notes: (1) Derived from Station 1 and Station 3 data.
Runoff Conc. = (comb. sewage load rate-san. sewage load rate)/ (comb. sewage flow rate-san. sewage flow rate)

TABLE C2.4

POLLUTANT RATIOS
LEAD vs SUSPENDED SOLIDS IN RUNOFF (1)

Date & Time	Lead (ug/l)	SS (mg/l)	Date & Time	Lead (ug/l)	SS (mg/l)
830519			830704		
14:15	280.0	218.0	20:00	170.0	135.0
14:45	290.0	269.0	20:15	170.0	126.0
15:15	160.0	263.0	20:30	180.0	135.0
15:45	90.0	164.0	20:45	180.0	131.0
16:15	60.0	76.2			
16:45	60.0	70.0			
17:25	60.0	47.3			
18:05	60.0	46.7			
830808			830822		
18:20	500.0	271.0	03:25	530.0	465.0
18:50	410.0	103.0	04:25	420.0	609.0
19:20	430.0	214.0	05:25	110.0	110.0
19:50	380.0	150.0	06:25	70.0	124.0
			07:25	50.0	44.1
			07:50	40.0	59.2
830916			831013		
08:10	220.0	294.0	15:00	230.0	525.0
08:55	130.0	126.0	16:00	130.0	105.0
09:40	80.0	106.0	17:00	100.0	71.6
10:30	160.0	114.0	18:00	80.0	50.3
11:10	90.0	106.0			
11:50	220.0	288.0			

Mean Ratio = 1.125 ug/mg
Std Dev. = 0.676 ug/mg

Notes: (1) Derived from Station 1 and Station 3 data.
Runoff Conc. = (comb. sewage load rate-san. sewage load rate)/(comb. sewage flow rate-san. sewage flow rate)

TABLE C 2.5

POLLUTANT RATIOS
ZINC vs SUSPENDED SOLIDS IN RUNOFF (1)

Date & Time	Zinc (ug/l)	SS (mg/l)	Date & Time	Zinc (ug/l)	SS (mg/l)
830519			830704		
14:15	463	260.1	20:00	331	176.3
14:45	405	290.4	20:15	272	151.3
15:15	254	283.5	20:30	295	159.6
15:45	177	185.3	20:45	373	161.8
16:15	142	88.4			
16:45	121	78.8			
17:25	93	56.4			
18:05	107	54.4			
830808			830822		
18:20	734	283.6	03:25	791	528.4
18:50	634	120.8	04:25	806	683.2
19:20	945	257.3	05:25	178	119.3
19:50	750	201.3	06:25	142	144.8
			07:25	151	54.4
			07:50	123	76.1
830916			831013		
08:10	507	361.3	15:00	413	588.2
08:55	312	156.4	16:00	177	119.9
09:40	248	127.8	17:00	133	80.2
10:30	284	137.0	18:00	98	64.1
11:10	224	128.9			
11:50	542	365.2			

Mean Ratio = 1.886 ug/mg
Std. Dev. = 0.909 ug/mg

Notes: (1) Derived from station 1 and station 3 data.
(2) Runoff Conc.=(Comb.sewage load rate - San.sewage load rate.)/(Comb.sewage flow rate - San.sewage flow rate.)

TABLE C 2.6

POLLUTANT RATIOS
COPPER vs SUSPENDED SOLIDS IN RUNOFF (1)

Date & Time	Copper (ug/l)	SS (mg/l)	Date & Time	Copper (ug/l)	SS (mg/l)
830519			830704		
14:15	160	260.1	20:00	108	176.3
14:45	182	290.4	20:15	76	151.3
15:15	95	283.5	20:30	113	159.6
15:45	58	185.3	20:45	382	161.8
16:15	32	88.4			
16:45	36	78.8			
17:25	41	56.4			
18:05	43	54.4			
830808			830822		
18:20	129	283.6	03:25	331	528.4
18:50	125	120.8	04:25	295	683.2
19:20	124	257.3	05:25	62	119.3
19:50	93	201.3	06:25	32	114.8
			07:25	12	54.4
			07:50	19	76.1
830916			831013		
08:10	361	258.3	15:00	160	588.2
08:55	156	172.4	16:00	46	119.9
09:40	128	147.9	17:00	37	80.2
10:30	137	160.0	18:00	20	64.1
11:10	129	111.4			
11:50	365	400.9			

Mean Ratio = 0.645 ug/mg
Std. Dev. = 0.422 ug/mg

Notes :(1) Derived from station 1 and station 3 data.
(2) Runoff Conc.= (Comb.sewage load rate - San.sewage load rate)/
(Comb.sewage flow rate - San.sewage flow rate)

TABLE C 2.7

POLLUTANT RATIOS
CADMIUM vs SUSPENDED SOLIDS IN RUNOFF (1)

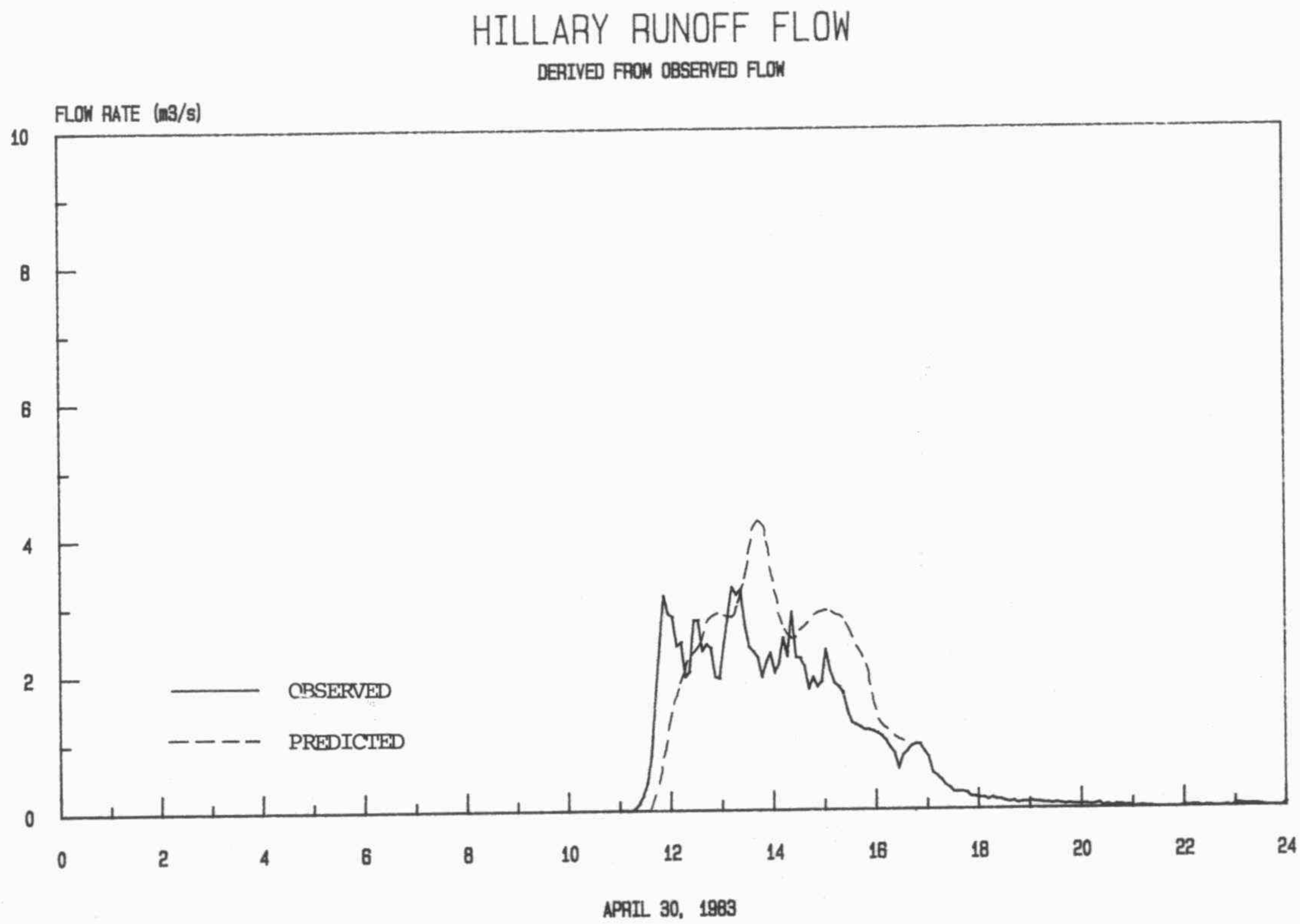
Date & Time	Cadmium (ug/l)	SS (mg/l)	Date & Time	Cadmium (ug/l)	SS (mg/l)
830519			830704		
14:15	9	260.0	20:00	2	176.2
14:45	10	290.4	20:15	3	151.2
15:15	10	283.5	20:30	3	159.5
15:45	10	185.3	20:45	3	161.8
16:15	9	88.4			
16:45	10	78.8			
17:25	9	56.4			
18:05	9	54.4			
830808			830822		
18:20	5	283.6	03:25	3	528.4
18:50	3	120.8	04:25	4	683.2
19:20	3	257.2	05:25	3	119.3
19:50	2	201.2	06:25	2	144.8
			07:25	2	54.4
			07:50	1	76.1
830916			831013		
08:10	4	361.3	15:00	4	588.2
08:55	4	156.3	16:00	4	119.9
09:40	4	127.7	17:00	4	80.2
10:30	4	137.0	18:00	3	64.1
11:10	4	128.8			
11:50	4	365.1			

Mean Ratio = 0.038 ug/mg
Std. Dev. = 0.042 ug/mg

Notes :(1) Derived from station 1 and station 3 data.
(2) Runoff Conc.= (Comb. sewage load rate - San. sewage load rate)/ (Comb. sewage flow rate - San. sewage flow rate)

APPENDIX C3

CALIBRATION HYDROGRAPHS



HILLARY RUNOFF FLOW
DERIVED FROM OBSERVED FLOW
FLOW RATE (m3/s)
10
8
6
4
2
0
OBSERVED
PREDICTED
0
2
4
6
8
10
12
14
16
18
20
22
24
APRIL 30, 1983

FIGURE C3.1

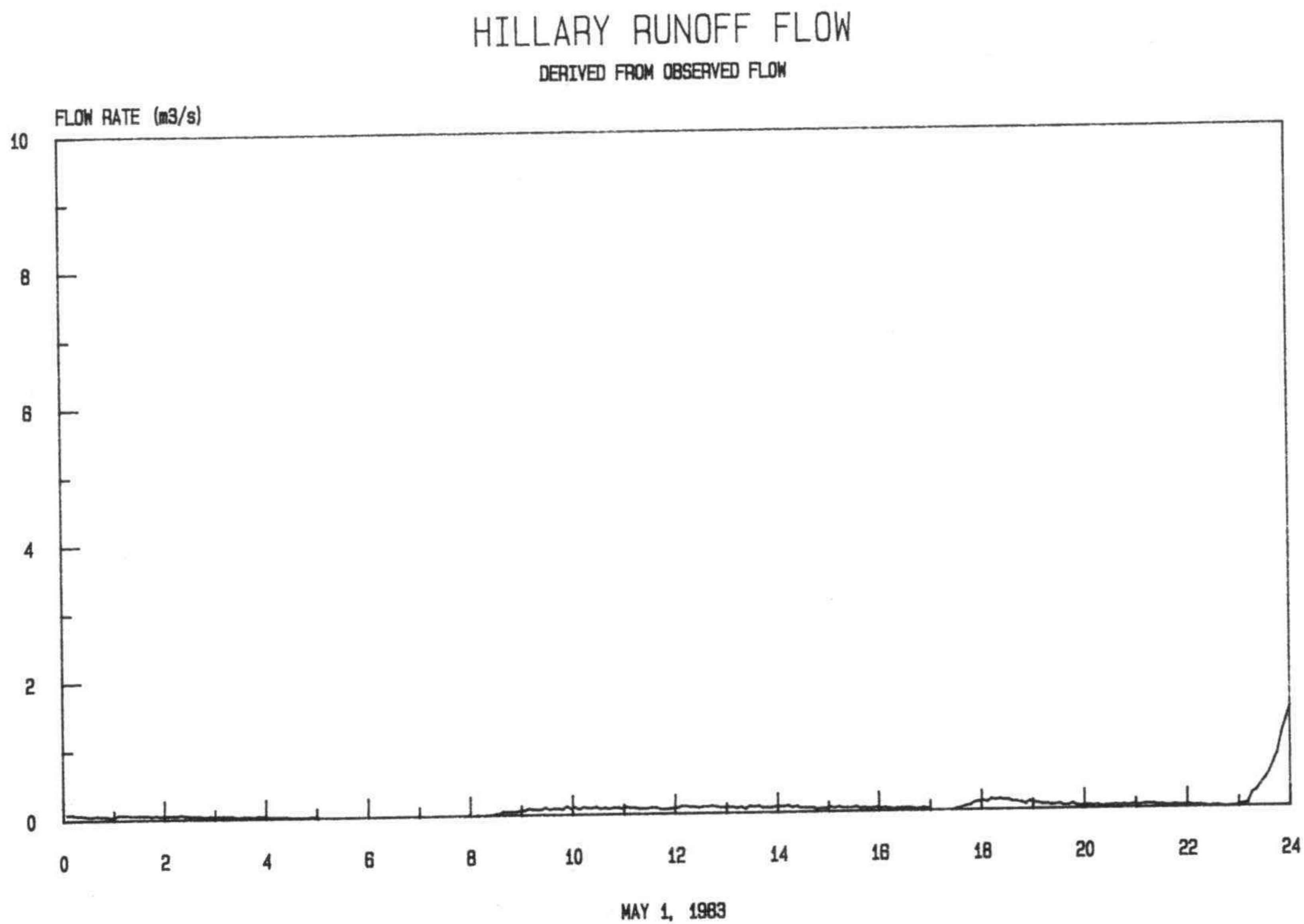

HILLARY RUNOFF FLOW
DERIVED FROM OBSERVED FLOW
FLOW RATE (m3/s)
10
8
6
4
2
0
0
2
4
6
8
10
12
14
16
18
20
22
24
MAY 1, 1983

FIGURE C3.2

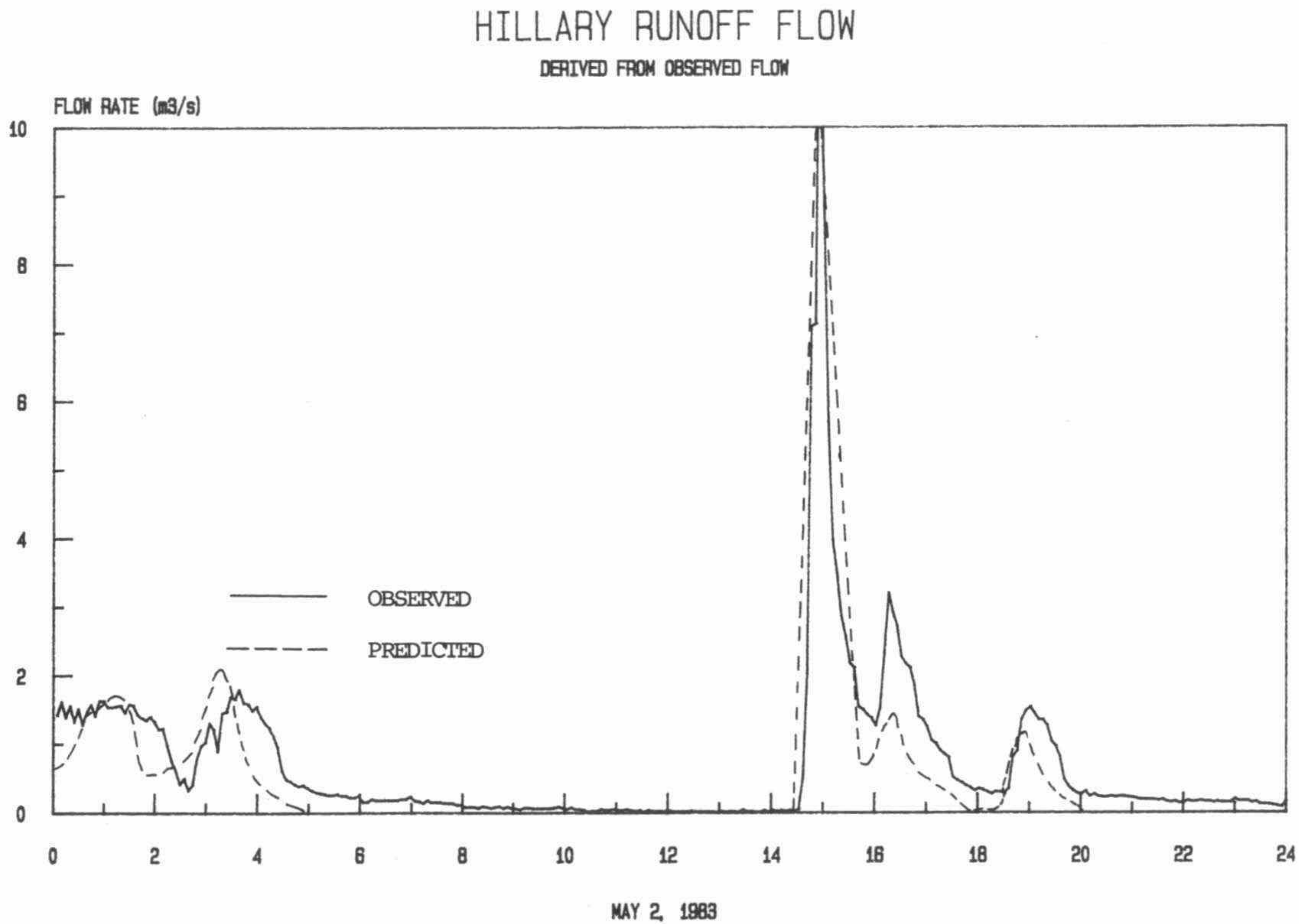

HILLARY RUNOFF FLOW
DERIVED FROM OBSERVED FLOW
FLOW RATE (m3/s)
10
8
6
4
2
0
OBSERVED
PREDICTED
0
2
4
6
8
10
12
14
16
18
20
22
24
MAY 2, 1983

FIGURE C3.3

HILLARY RUNOFF FLOW

DERIVED FROM OBSERVED FLOW

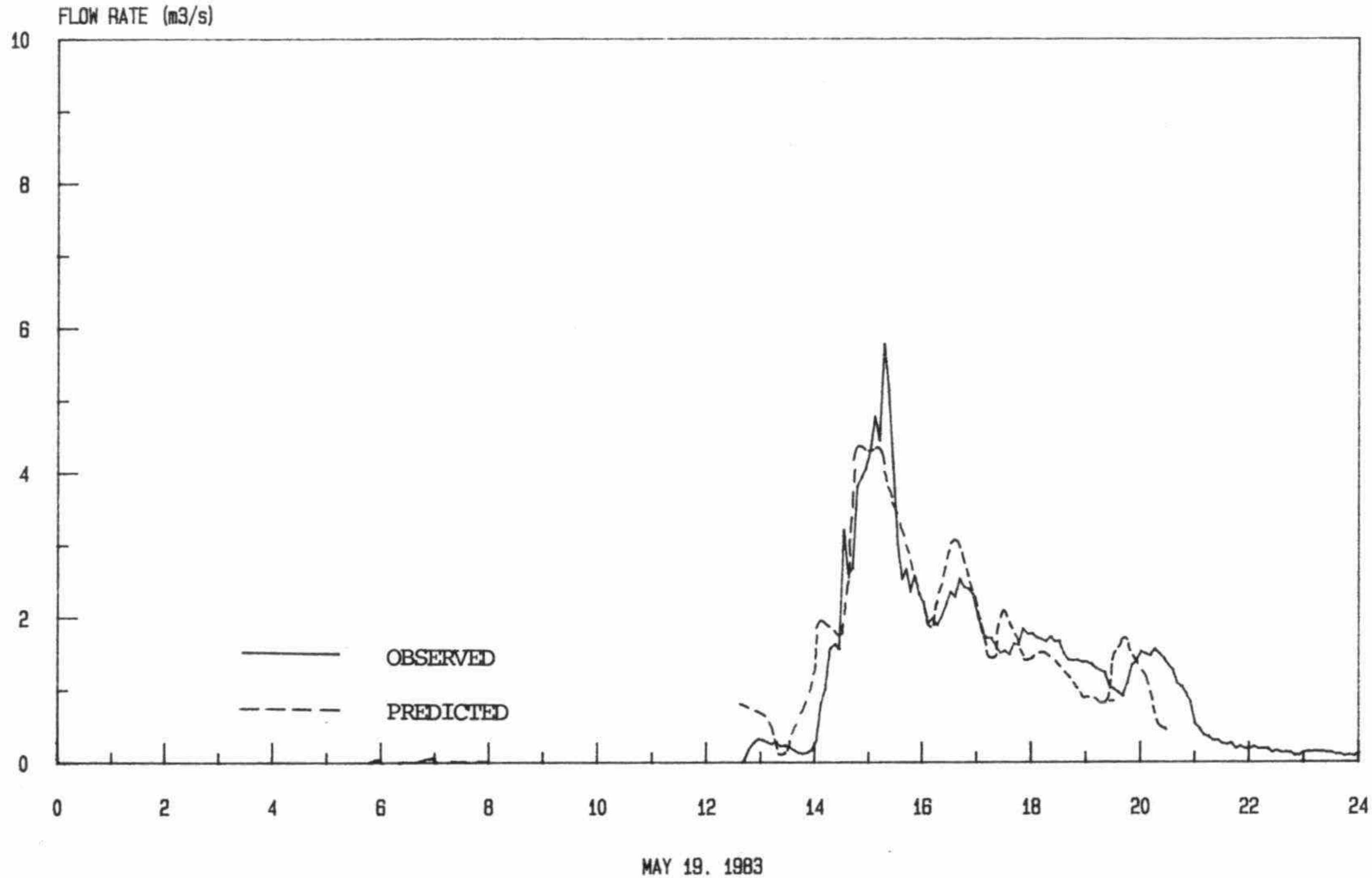


FIGURE C3.4

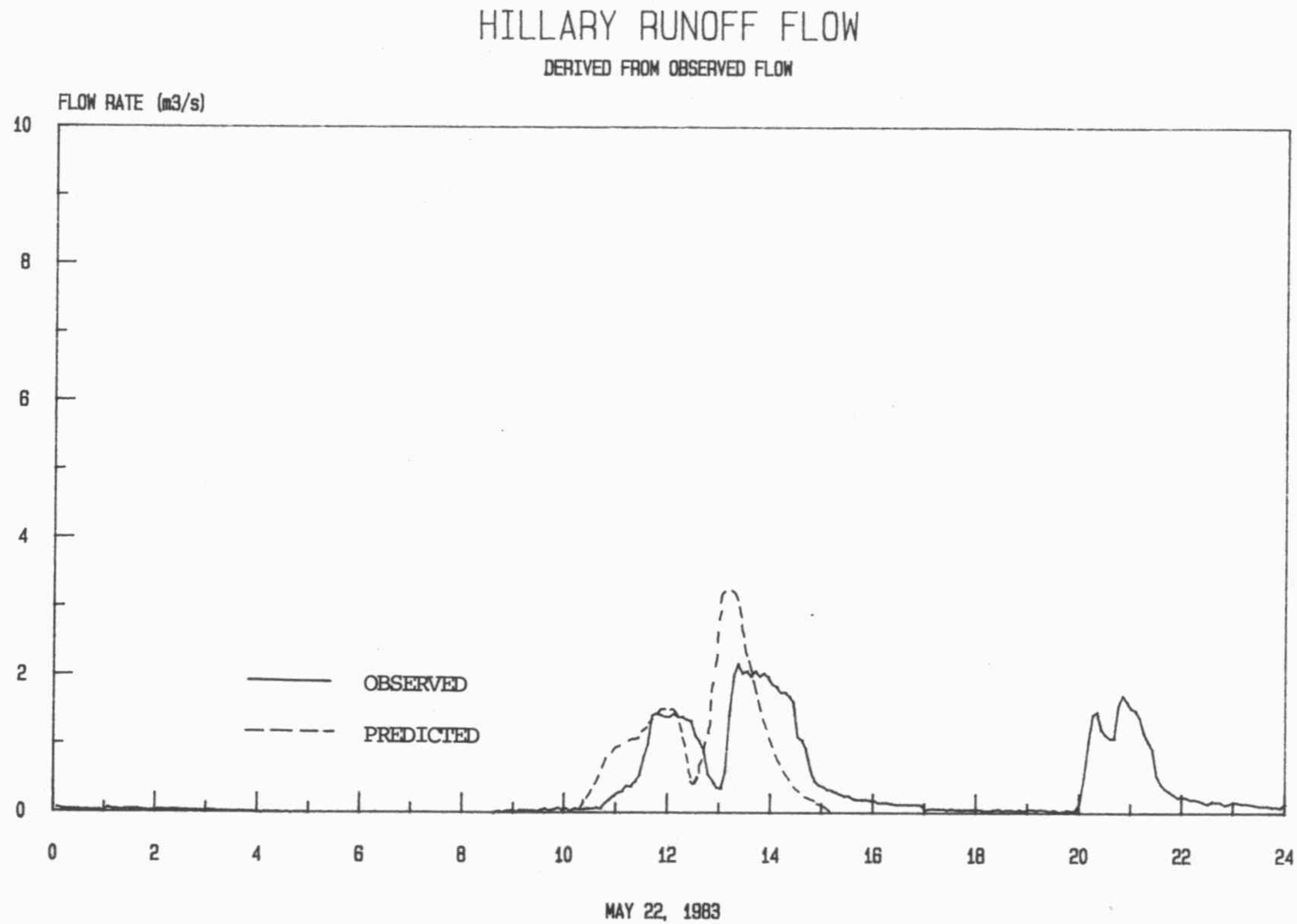

HILLARY RUNOFF FLOW
DERIVED FROM OBSERVED FLOW
FLOW RATE (m3/s)
10
8
6
4
2
0
OBSERVED
PREDICTED
0
2
4
6
8
10
12
14
16
18
20
22
24
MAY 22, 1983

FIGURE C3.5

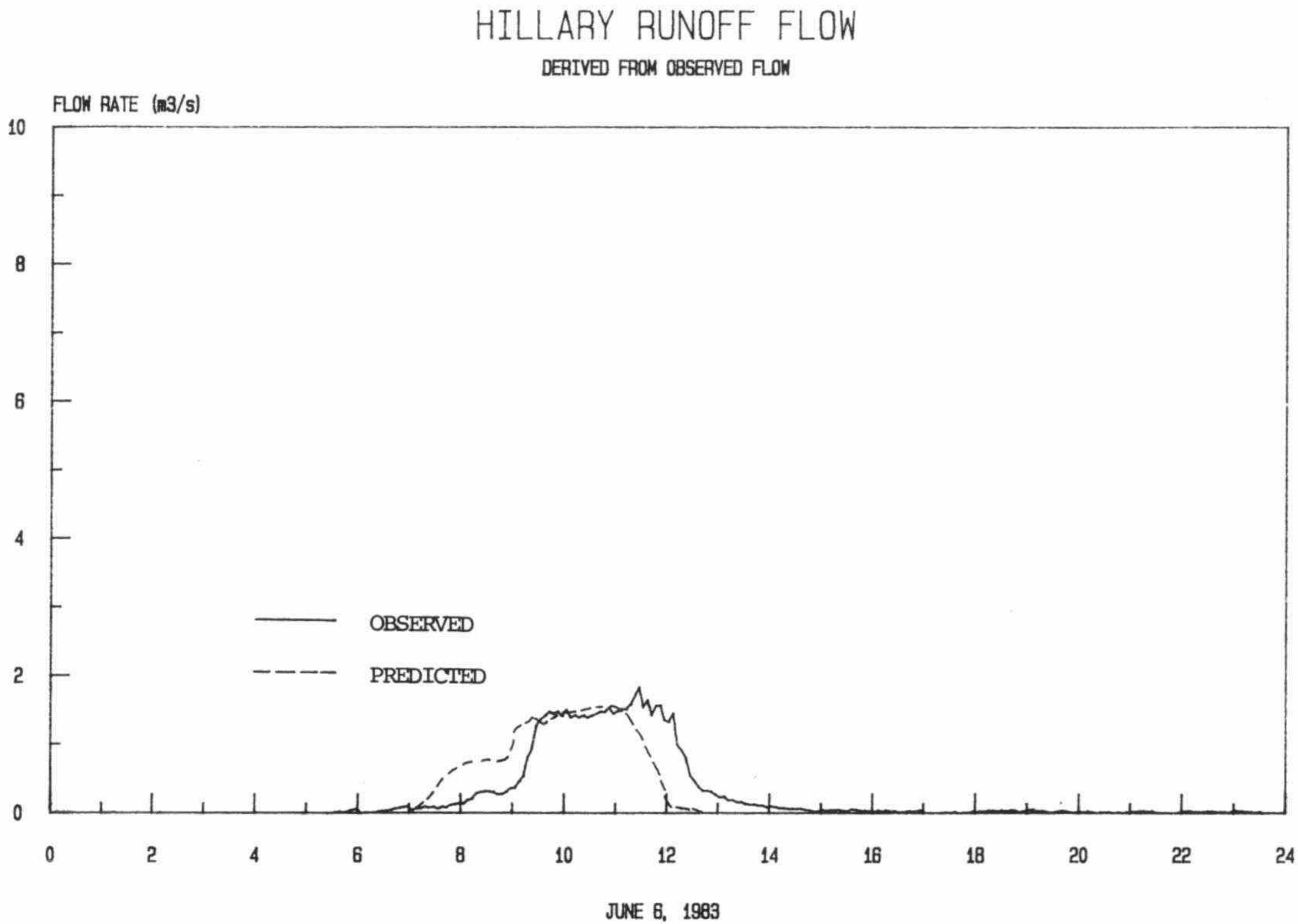

HILLARY RUNOFF FLOW
DERIVED FROM OBSERVED FLOW
FLOW RATE (m3/s)
10
8
6
4
2
0
OBSERVED
PREDICTED
0
2
4
6
8
10
12
14
16
18
20
22
24
JUNE 6, 1983

FIGURE C3.6

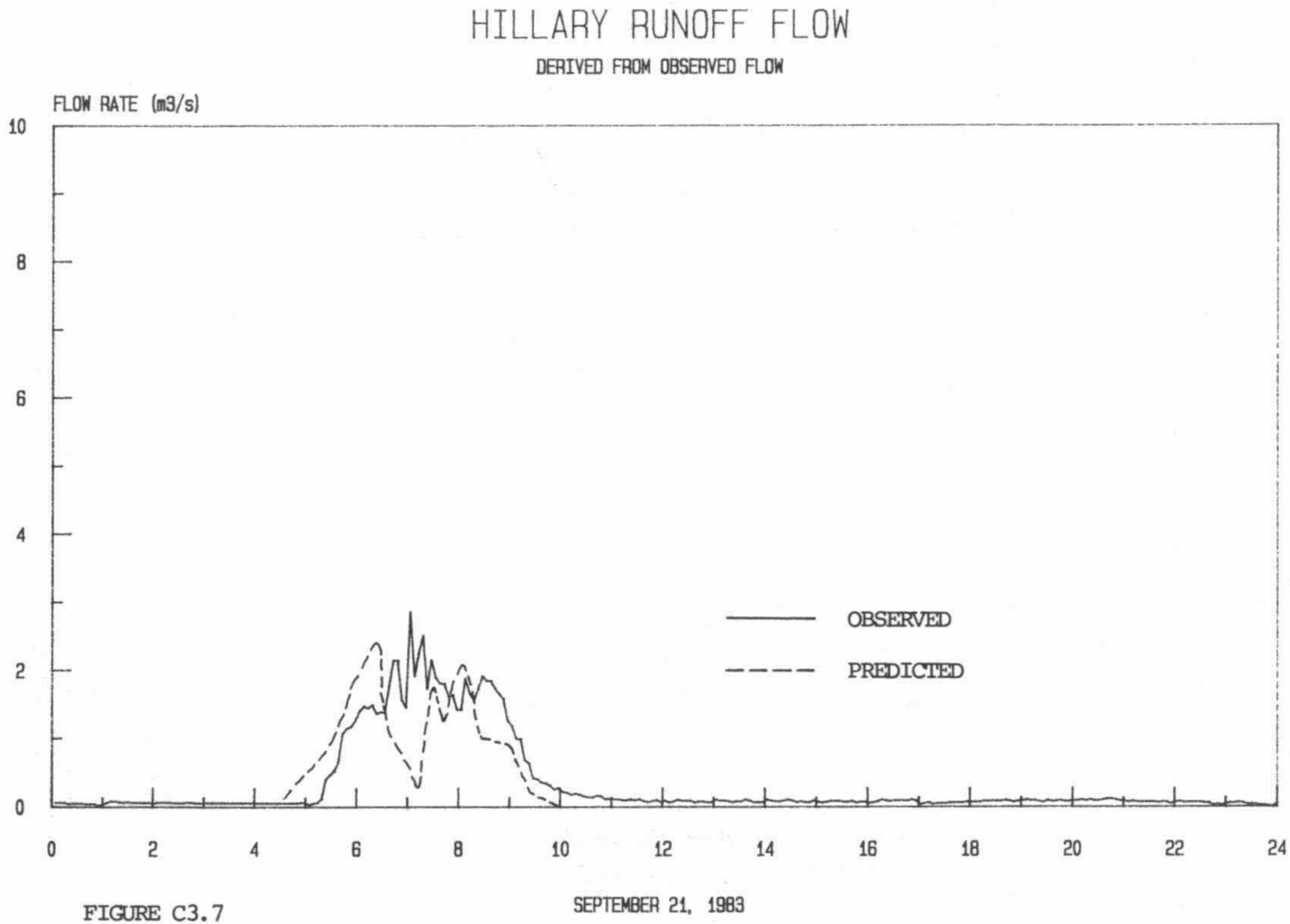

HILLARY RUNOFF FLOW
DERIVED FROM OBSERVED FLOW
FLOW RATE (m3/s)
10
8
6
4
2
0
0
2
4
6
8
10
12
14
16
18
20
22
24
OBSERVED
PREDICTED
SEPTEMBER 21, 1983

FIGURE C3.7

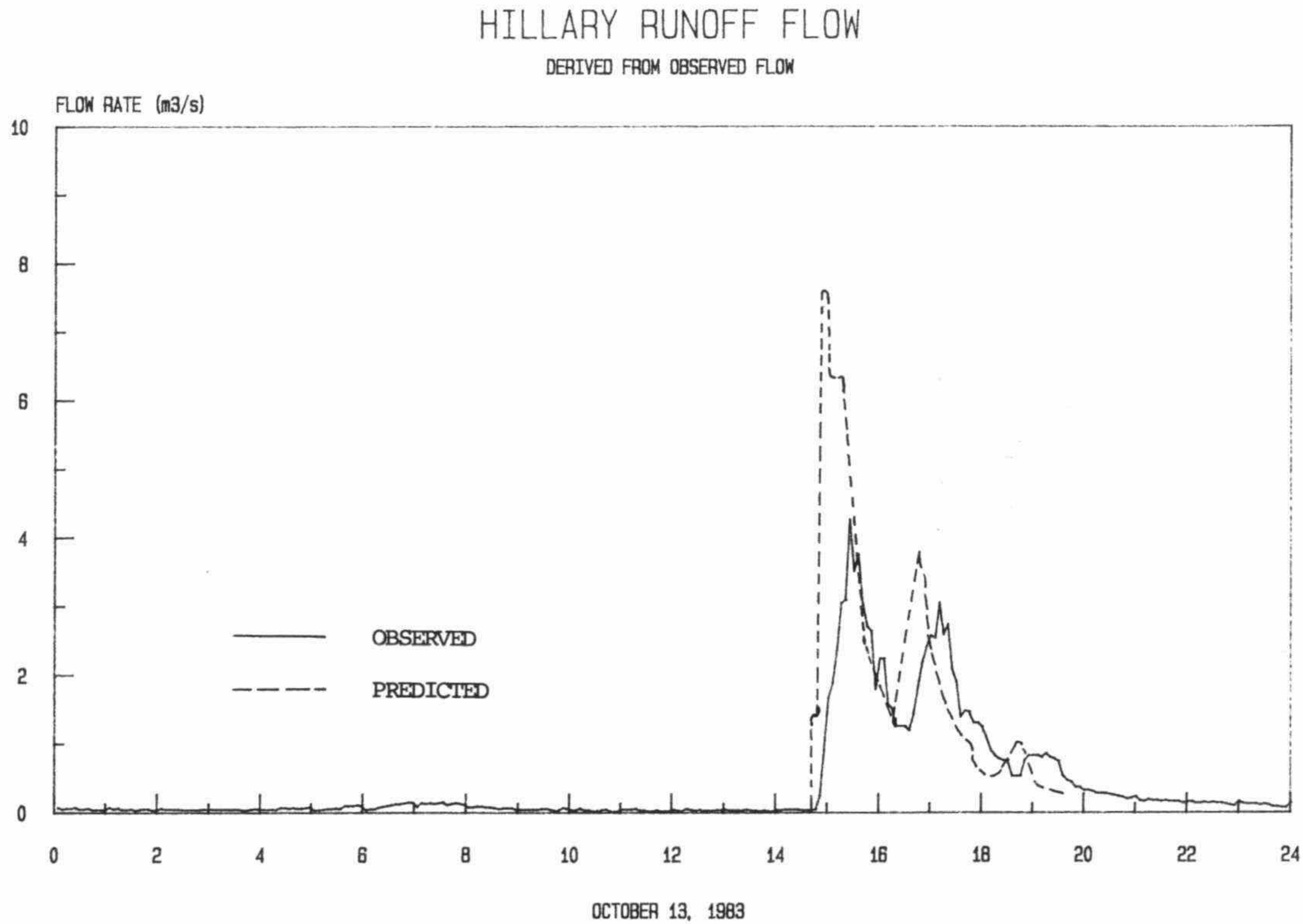

HILLARY RUNOFF FLOW
DERIVED FROM OBSERVED FLOW
FLOW RATE (m3/s)
10
8
6
4
2
0
OBSERVED
PREDICTED
0
2
4
6
8
10
12
14
16
18
20
22
24
OCTOBER 13, 1983

FIGURE C3.8

APPENDIX D1

BASE CASE RESULTS

TABLE D1.1

SUMMARY OF RESULTS FOR APRIL - OCTOBER 1979
EXISTING SYSTEM

	MHID	New+Old Tank Size	Volume m^3	SS kg	BOD5 kg	SOL-P kg	Tp kg	Cd kg	Cu kg	Pb kg	Zn kg
Overflow	BC Gp		493145	96554	23150	324	1015	3.975	61.088	101.070	175.584
Stored	Hyde	7823	110005	15943	4655	75	211	.816	12.753	20.180	34.927
Overflow	660		11974	523	556	8	15	.039	.960	.140	.510
Overflow	670		92652	4214	4335	62	122	.310	7.780	1.280	4.320
Overflow	6361		384718	80343	18370	244	808	3.223	50.176	85.753	147.880
Stored		8823	216421	32156	9667	155	386	1.472	22.800	35.389	61.687
Overflow	6362		331356	71267	15782	207	704	2.822	43.798	81.411	130.293
Stored		12823	544745	41232	12066	192	490	1.872	29.178	31.774	79.275
Overflow	6363		291356	64111	13974	181	624	2.508	38.741	89.080	116.202
Stored		16823	309784	48389	13873	218	570	2.186	34.236	24.510	93.365
Overflow	6364		255168	57257	12322	157	550	2.211	33.922	96.208	102.702
Stored		20823	345972	55242	15526	242	644	2.484	39.055	18.224	106.860
Overflow	6365		221572	50649	10770	135	481	1.939	29.533	52.156	90.416
Stored		24823	379567	61850	17078	264	713	2.755	43.444	68.988	119.151
Overflow	6366		193903	45170	9490	117	430	1.742	26.425	47.185	81.856
Stored		28823	407236	67330	18358	282	764	2.952	46.552	73.959	127.711
Overflow	6367		175623	41487	8598	105	394	1.599	24.150	43.470	75.487
Stored		32823	425516	71012	19206	294	801	3.096	48.827	77.673	134.081
Overflow	6368		127623	31005	6295	75	290	1.185	17.549	32.397	56.550
Stored		44823	473516	81494	21509	324	904	3.509	55.429	88.746	153.069
Overflow	6369		116493	28379	5746	68	269	1.106	16.341	30.407	53.069
Stored		48823	601139	84119	22058	331	924	3.588	56.636	90.737	156.497

Note: Event 790711 included.Tank size in m3.

TABLE D1.2

EVENT 790711 RESULTS
EXISTING SYSTEM

	MHID	Old+New Tank Size	Volume m^3	SS kg	BOD5 kg	SOL-P kg	Tp kg	Cd kg	Cu kg	Pb kg	Zn kg
Overflow	BC Gp		157493	33914	7151	95	325	1.331	19.857	36.380	63.143
Stored	Hyde	7823	7823	193	106	3	3	.011	.145	.165	.274
Overflow	660		11974	523	556	8	15	.039	.960	.140	.510
Overflow	670		92650	4214	4335	62	122	.310	7.780	1.280	4.320
Overflow	6361		156493	33820	7067	92	324	1.325	19.789	36.359	63.065
Stored		8823	8823	287	189	6	5	.016	.212	.186	.353
Overflow	6362		152493	33635	6989	89	321	1.317	19.647	36.172	62.767
Stored		12823	12823	471	268	9	8	.024	.354	.373	.651
Overflow	6363		148493	33373	6899	87	318	1.305	19.452	35.882	62.310
Stored		16823	16823	734	357	11	11	.036	.550	.663	1.107
Overflow	6364		144493	33023	6797	84	315	1 .290	9.203	35.486	61.680
Stored		20823	20823	1084	460	14	14	.051	.799	1.059	1.738
Overflow	6365		140493	32584	6680	82	310	1.272	18.905	34.991	60.877
Stored		24823	24823	1523	577	16	19	.069	1.097	1.554	2.540
Overflow	6366		136493	32060	6551	80	305	1.251	18.563	34.406	59.913
Stored		28823	28823	2047	706	19	24	.090	1.439	2.139	3.505
Overflow	6367		132493	31458	6410	77	299	1.227	18.183	33.742	58.799
Stored		32823	32823	2648	847	21	30	.114	1.819	2.803	4.619
Overflow	6368		120493	29238	5925	70	278	1.139	16.843	31.329	54.670
Stored		44823	44823	4869	1332	28	51	.202	3.159	5.216	8.748
Overflow	6369		116493	28379	5746	68	269	1.106	16.341	30.407	53.069
Stored		48823	48823	5727	1511	30	59	.235	3.661	6.138	10.348

Note:(1) Tank size in m3.

TABLE D1.3

FECAL COLIFORM LOADS - EXISTING SYSTEM

Event Date	Overflow F. Coliform Load (10^{12} Organisms)
790413	276
790426	385
790503	130
790525	212
790610	670
790629	123
790710	87
790716	341
790725	288
790731	642
790801	340
790914	639
791008	141
790715 (1)	81
790807 (1)	135
790826 (1)	85

NOTE:

(1) Storm 790715 represents 5 storms of 4-6 mm precip.
Storm 790807 represents 3 storms of 6-8 mm precip.
Storm 790826 represents 5 storms of 8-10 mm precip.
F.C. Load is for single storm.

APPENDIX D2

BREAKDOWN OF ESTIMATES

ORDER OF COSTS

Unit Costs (Braganza, 1985)

1.	Bulk excavation and backfill		\$ 2.7/m³
2.	Extra over item (1) for cartaway		\$ 5.0/m³
3.	Reinforced concrete including reinforcing steel and formwork		\$ 200/m³
4.	Road pavement, wearing course, basecourse and subbase		\$ 50/m²
5.	Excavate, supply and lay 1.2m die concrete sewer pipe, 5m depth		\$ 450/m
6.	Excavate, supply and lay 2.0m doa concrete sewer pipe, 5m depth		\$ 780/m
7.	Local detention tank:		
	2.5m dia pipe	\$ 1,148/m	
	Road pavement 5.5m wide	275/m	
		\$ 1,423/m	
	60m long pipe at \$1,423	\$87,380	
	Access manhole (say)	4,000	
		\$91,380	\$91,380/No.

(1) 16,000 m^3 Tank at Hyde Ave

Item	Quantity	Unit	Rate	Amount
Excavate, cartaway	26,820	m^3	5.00	134,000
Concrete	3,952	m^3	200.00	790,000
Total of (1)				924,000

(2) 41,000 m^3 Tank at Black Creek

Excavate, replace	95,000	m^3	2.70	257,000
Extra over for cartaway	43,000	m^3	5.00	215,000
Concrete	8,766	m^3	200.00	1,753,000
Total of (2)				2,225,000

(3) 35,000 m^3 Tank at Black Creek

Excavate, replace	80,820	m^3	2.70	218,000
Extra over for cartaway	41,450	m^3	5.00	207,000
Concrete	7,289	m^3	200.00	1,458,000
Total of (3)				1,883,000

(4) 15,000 m^3 Tank at Black Creek

Excavate, replace	37,539	m^3	2.70	101,000
Extra over for cartaway	20,529	m^3	5.00	103,000
Concrete	3,305	m^3	200.00	661,000
Total of (4)				865,000

(5) 4,000 m^3 Tank at Berry Road

Extrapolate from Hyde Ave Tank

924,000 x 4,000/16,000 = 222,000

Add 10% for small job	22,000	244,000
Total of (5)		244,000

(6) 2m dia Pipe between New Hyde Ave Tank and New Black Creek Tank

Excavate, supply and lay pipe	1,500	m	780.00	1,170,000
Road reinstatement, 7m wide	10,500	m^2	50.00	525,000
Total of (6)				1,695,000

(7) 1.2 dia Pipe for NEW
Black Creek Sewer

Excavate, supply and lay pipe	2,100 m	450.00	945,000
Total of (7)			945,000

APPENDIX D3

CSO CONTROL SCHEMES RESULTS

TABLE D3.1

REQUIRED CAPACITIES OF CONNECTOR SEWERS

Connector Sewer (1)				Required Capacity (m³/s)
	P21	P31	P41	7.5
P11	P22	P32	P42	1.4
P12	P23	P33	P45	3.3
P13	P24	P34	P43	2.3
P14	P25	P35		7.0
			P44	3.7
	P26			14.5

NOTE:

(1) See Figure 7.1 for naming of connector sewers.

TABLE D 3.2

SUMMARY OF RESULTS FOR APRIL - OCTOBER 1979
1983 CATCHMENTS AND REGULATORS RESET

	MHID	Old+New Tank Size	Volume m^3	SS kg	BOD5 kg	SOL-P kg	TP kg	Cd kg	Cu kg	Pb kg	Zn kg
Overflow	BC Gp		221154	44413	9266	134	444	1.778	29.006	47.692	83.970
Stored	Hyde	7823	110005	15874	4635	75	210	0.811	12.711	20.096	34.791
Overflow	660		17035	790	788	12	22	0.057	1.382	0.255	0.824
Overflow	670		114264	5746	5474	80	156	0.406	9.850	2.039	6.301
Overflow	6361		207766	41489	8594	123	409	1.641	26.904	44.518	78.394
Stored		8823	123395	18802	5363	86	232	0.897	14.228	22.718	39.165
Overflow	6362		179869	36099	7379	106	350	1.405	23.039	38.105	67.496
Stored		12823	151292	24193	6590	104	291	1.131	18.093	29.132	50.063
Overflow	6363		157137	31310	6373	91	301	1.208	19.753	32.613	58.183
Stored		16823	174024	28981	7610	118	339	1.329	21.379	34.624	59.377
Overflow	6364		143546	28409	5788	84	271	1.090	17.789	29.332	52.611
Stored		20823	187615	31882	8208	126	368	1.447	23.343	37.904	64.948
Overflow	6365		131546	25809	5269	77	246	0.991	16.137	26.590	47.921
Stored		24823	199615	34483	8632	133	393	1.545	24.995	40.646	69.638
Overflow	6366		119546	23137	4736	69	221	0.886	14.403	23.688	42.946
Stored		28823	211615	37154	9167	141	419	1.650	26.729	43.548	74.613
Overflow	6367		109813	20917	4301	63	198	0.798	12.932	21.221	38.704
Stored		32823	221347	39375	9601	146	442	1.739	28.201	46.016	78.855
Overflow	6368		92180	17108	3554	53	163	0.654	10.578	17.305	31.781
Stored		44823	238981	43183	10348	157	477	1.882	30.554	49.932	85.778
Overflow	6369		88180	16334	3395	51	156	0.624	10.111	16.530	30.346
Stored		48823	242981	43957	10507	159	484	1.912	31.021	50.707	87.214

Notes: (1) Value of Event 790711 included.
(2) Tank size in m3.

TABLE D 3.3

EVENT 790711 RESULTS
1983 CATCHMENTS & RESET REGULATORS

	MHID	Old+New Tank Size	Volume m^3	SS kg	BOD5 kg	SOL-P kg	TP kg	Cd kg	Cu kg	Pb kg	Zn kg
Overflow	BC Gp		129180	21350	4548	76	205	0.819	13.350	21.570	39.510
Stored	Hyde	7823	7823	124	86	3	2	0.006	0.103	0.081	0.138
Overflow	660		16838	771	772	12	22	0.056	1.355	0.246	0.799
Overflow	670		109524	5293	5103	75	146	0.378	9.191	1.836	5.741
Overflow	6361		128180	21261	4499	74	204	0.815	13.274	21.507	39.381
Stored		8823	8823	213	136	6	4	0.011	0.177	0.153	0.257
Overflow	6362		124180	21108	4444	72	202	0.808	13.150	21.353	39.127
Stored		12823	12823	366	191	8	6	0.017	0.301	0.307	0.511
Overflow	6363		120180	20878	4378	69	200	0.799	12.977	21.116	38.724
Stored		16823	16823	596	257	11	8	0.026	0.474	0.544	0.915
Overflow	6364		116180	20563	4299	67	196	0.786	12.754	20.792	38.154
Stored		20823	20823	911	337	13	11	0.039	0.697	0.868	1.484
Overflow	6365		112180	20160	4205	65	192	0.771	12.485	20.381	37.418
Stored		24823	24823	1315	430	15	15	0.054	0.966	1.279	2.220
Overflow	6366		108180	19672	4097	62	188	0.752	12.173	19.889	36.524
Stored		28823	28823	1802	538	18	20	0.073	1.278	1.771	3.114
Overflow	6367		104180	19123	3977	60	182	0.731	11.822	19.332	35.510
Stored		32823	32823	2352	658	20	26	0.094	1.629	2.328	4.128
Overflow	6368		92180	17108	3554	53	163	0.654	10.578	17.305	31.781
Stored		44823	44823	4366	1081	27	45	0.171	2.873	4.355	7.857
Overflow	6369		88180	16334	3395	51	156	0.624	10.111	16.530	30.346
Stored		48823	48823	5140	1240	29	52	0.201	3.340	5.130	9.293

Note: Tank size in m3.

TABLE D 3.4

SUMMARY OF RESULTS FOR APRIL - OCTOBER 1979
RUNOFF CONTROL SCHEME A

	MHID	Old+New Tank Size	Volume m^3	SS kg	BOD5 kg	SOL-P kg	TP kg	Cd kg	Cu kg	Pb kg	Zn kg
Overflow	BC Gp		280730	44509	11696	184.6	469.9	1.806	28.270	43.810	76.859
Stored	Hyde	7823	109079	13515	4355	77.2	194.1	0.740	11.452	17.655	30.751
Overflow	660		11269	493	531	7.7	14.7	0.036	0.918	0.125	0.476
Overflow	670		90257	4037	4260	60.7	119.7	0.302	7.593	1.127	4.011
Overflow	6361		223720	36503	8979	248.6	359.4	1.416	22.636	36.674	63.311
Stored		8823	166082	21523	7072	282.2	280.6	1.056	16.088	24.360	42.794
Overflow	6362		192372	31301	7523	187.0	298.2	1.179	19.019	30.721	53.058
Stored		12823	201430	26725	8528	343.7	341.8	1.293	19.850	30.314	53.046
Overflow	6363		156372	26292	6188	131.6	242.3	0.963	15.562	25.206	43.594
Stored		16823	233430	31733	9863	399.2	397.6	1.508	23.307	35.829	62.510
Overflow	6364		134627	22863	17299	99.1	208.8	0.834	13.546	22.103	38.221
Stored		20823	255175	35162	10751	431.6	431.2	1.636	25.322	38.933	67.884
Overflow	6365		118627	20441	4661	79.7	187.5	0.752	12.261	20.145	34.818
Stored		24823	271175	37585	11390	451.0	452.4	1.718	26.608	40.889	71.287
Overflow	6366		105650	18403	4161	66.3	171.6	0.693	11.302	18.646	32.227
Stored		28823	284152	39622	11890	464.4	468.4	1.778	27.567	42.389	73.877
Overflow	6367		93894	16472	3705	55.9	158.0	0.636	10.399	17.208	29.745
Stored		32823	295908	41554	12346	475.8	483.0	1.834	28.469	43.827	76.359
Overflow	6368		79520	14078	3159	46.8	136.3	0.522	9.060	15.021	25.912
Stored		44823	310283	43948	12892	483.9	503.7	1.919	29.809	46.013	80.193
Overflow	6369		75520	13356	2997	44.4	129.3	0.523	8.602	14.257	24.584
Stored		48823	314283	44669	13053	486.3	510.7	1.947	30.266	46.777	81.521

Note:(1) 1983 sewer system. Event 790711 included.
(2)Tank size in m3.

TABLE D 3.5

EVENT 790711 RESULTS
RUNOFF CONTROL SCHEME A (1)

	MHID	Old+New Tank Size	Volume m^3	SS kg	BOD5 kg	SOL-P kg	TP kg	Cd kg	Cu kg	Pb kg	Zn Kg
Overflow	BC Gp		116520	18301	4248	69.6	179.9	0.721	11.888	19.439	33.483
Stored	Hyde	7823	7823	273	139	5.2	4.1	0.015	0.242	0.255	0.393
Overflow	660		11269	493	531	7.7	14.7	0.036	0.918	0.125	0.476
Overflow	670		90257	4037	4260	60.7	119.7	0.302	7.593	1.127	4.011
Overflow	6361		115519	18330	4218	68.6	179.4	0.723	11.929	19.539	33.593
Stored		8823	8823	245	169	6.2	4.6	0.015	0.199	0.155	0.283
Overflow	6362		115519	18160	4149	66.0	177.2	0.715	11.785	19.363	33.321
Stored		12823	12823	415	238	8.7	6.8	0.023	0.344	0.331	0.555
Overflow	6363		107519	17911	4069	63.6	174.3	0.704	11.586	19.092	32.893
Stored		16823	16823	664	318	11.2	9.6	0.033	0.542	0.602	0.983
Overflow	6364		103519	17576	3975	61.1	170.8	0.690	11.337	18.728	32.300
Stored		20823	20823	998	411	13.6	13.2	0.047	0.791	0.967	1.576
Overflow	6365		99519	17159	3868	58.7	166.5	0.673	11.044	18.278	31.547
Stored		24823	24823	1416	519	16.0	17.4	0.064	1.085	1.416	2.329
Overflow	6366		95519	16664	3748	56.3	161.6	0.654	10.710	17.750	30.650
Stored		28823	28823	1910	639	18.4	22.4	0.084	1.419	1.944	3.226
Overflow	6367		91519	16103	3616	53.9	156.0	0.631	10.339	17.152	29.625
Stored		32823	32823	2472	771	20.8	28.0	0.106	1.789	2.543	4.251
Overflow	6368		79520	14078	3159	46.8	136.3	0.552	9.060	15.021	25.912
Stored		44823	44823	4497	1228	27.9	47.7	0.186	3.069	4.673	7.964
Overflow	6369		75520	13356	2997	44.4	129.3	0.523	8.602	14.257	24.584
Stored		48823	48823	5218	1389	30.3	54.7	0.214	3.526	5.437	9.292

Notes: (1) 1983 sewer system.
(2) Tank size in m3.

TABLE D 3.6

SUMMARY OF RESULTS FOR APRIL - OCTOBER 1979
RUNOFF CONTROL SCHEME B (1)

	MHID	Old+New Tank Size	Volume m^3	SS kg	BOD5 kg	SOL-P kg	TP kg	Cd kg	Cu kg	Pb kg	Zn Kg
Overflow	BC Gp		230800	35698	9581	153	377	1.458	22.538	36.512	61.995
Stored	Hyde	7823	88367	10362	3468	63	153	0.576	8.858	13.496	23.510
Overflow	660		11101	490	526	59	15	0.036	.909	0.127	0.471
Overflow	670		89518	4028	4245	158	119	0.303	7.531	1.155	4.041
Overflow	6361		176357	28298	7032	92	279	1.105	17.559	29.799	49.597
Stored		8823	142804	17762	6016	123	236	0.880	13.347	20.012	35.225
Overflow	6362		144743	23686	5741	89	224	0.891	14.139	24.345	40.261
Stored		12823	174417	22373	7307	126	291	1.094	16.737	25.468	44.561
Overflow	6363		119885	19895	4731	73	183	0.738	11.730	20.552	33.770
Stored		16823	199275	26164	8317	142	331	1.246	19.177	29.260	51.051
Overflow	6364		105949	17778	4187	63	163	0.660	10.476	18.552	30.391
Stored		20823	213211	28282	8861	152	351	1.325	20.430	31.260	54.430
Overflow	6365		93949	15923	3735	56	149	0.604	9.594	17.118	27.983
Stored		24823	225211	30136	9314	159	366	1.380	21.313	32.695	56.829
Overflow	6366		82685	14140	3304	49	136	0.551	8.752	15.730	25.654
Stored		28823	236475	31921	9745	166	379	1.433	22.155	34.081	59.167
Overflow	6367		76367	13159	3072	45	128	0.521	8.296	14.966	24.377
Stored		32823	242793	32901	9976	170	387	1.462	22.610	34.847	60.444
Overflow	6368		64368	11069	2587	38	108	0.438	7.004	12.630	20.514
Stored		44823	254793	34991	10461	177	408	1.545	23.902	37.183	64.308
Overflow	6369		60368	10340	2419	36	100	0.410	6.551	11.807	19.163
Stored		48823	258793	35720	10629	179	415	1.574	24.356	38.006	65.658

Notes:(1) 1983 sewer system. Event 790711 included.
(2) Tank size in m3.

TABLE D 3.7

EVENT 790711 RESULTS
RUNOFF CONTROL SCHEME B (1)

	MHID	Old+New Tank Size	Volume m^3	SS kg	BOD5 kg	SOL-P kg	TP kg	Cd kg	Cu kg	Pb kg	Zn Kg
Overflow	BC Gp		101368	15640	3778	62	154	0.623	9.972	17.748	28.804
Stored	Hyde	7823	7823	354	161	5	5	0.017	0.304	0.405	0.556
Overflow	660		11101	490	526	8	15	0.036	0.909	0.127	0.471
Overflow	670		89518	4028	4245	60	119	0.303	7.531	1.155	4.014
Overflow	6361		100367	15714	3762	60	154	0.625	10.040	17.927	29.006
Stored		8823	8823	280	177	6	5	0.015	0.234	0.226	0.356
Overflow	6362		96367	15501	3682	57	152	0.616	9.867	17.673	28.646
Stored		12823	12823	493	257	9	7	0.024	0.407	0.481	0.716
Overflow	6363		92367	15196	3587	55	148	0.603	9.639	17.306	28.107
Stored		16823	16823	798	352	11	11	0.037	0.636	0.848	1.255
Overflow	6364		88367	14802	3478	52	144	0.587	9.361	16.840	27.393
Stored		20823	20823	1192	461	14	15	0.053	0.913	1.314	1.969
Overflow	6365		84367	14324	3355	50	139	0.568	9.043	16.289	26.521
Stored		24823	24823	1670	585	16	20	0.072	1.232	1.865	2.841
Overflow	6366		80367	13774	3219	48	134	0.546	8.685	15.659	25.510
Stored		28823	28823	2221	721	18	25	0.094	1.590	2.494	3.852
Overflow	6367		76367	13159	3072	45	128	0.521	8.296	14.966	24.377
Stored		32823	32823	2835	867	21	31	0.118	1.978	3.188	4.984
Overflow	6368		64368	11069	2587	38	108	0.438	7.004	12.630	20.514
Stored		44823	44823	4925	1352	28	52	0.201	3.270	5.524	8.848
Overflow	6369		60368	10340	2419	36	100	0.410	6.551	11.807	19.163
Stored		48823	48823	5654	1520	30	59	0.230	3.724	6.347	10.198

Notes: (1) 1983 sewer system.
(2) Tank size in m3.

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